

SCHOOL OF CIVIL ENGINEERING



# JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-80/7

GEOTECHNICAL DATA BANK  
FOR INDIANA

Y-K. T. Lo



PURDUE UNIVERSITY  
INDIANA STATE HIGHWAY COMMISSION



## Final Report

## GEOTECHNICAL DATA BANK FOR INDIANA

TO: H. L. Michael, Director  
Joint Highway Research Project

May 14, 1980

FROM: C. W. Lovell, Research Engineer  
Joint Highway Research Project

Project: C-36-51T

File: 1-5-20

Attached is the Final Report on the HPR Part I Study titled "A Computerized Information Storage and Retrieval System for the Soils of Indiana". The report is entitled "Geotechnical Data Bank for Indiana" and is authored by T-K. T. Lo of our staff.

The research involved the storage of about 7500 data sets and selected statistical interpretations of them. These included regression equations to predict soil properties and formulation of statistical soil profiles for a variety of soil areas in Indiana. Examples of how to use the data bank in routine ISHC operations were also developed and reported.

The Report is submitted for review, comment and acceptance in fulfillment of the objectives of the referenced HPR study.

Respectfully submitted,

*C. W. Lovell/HLM*

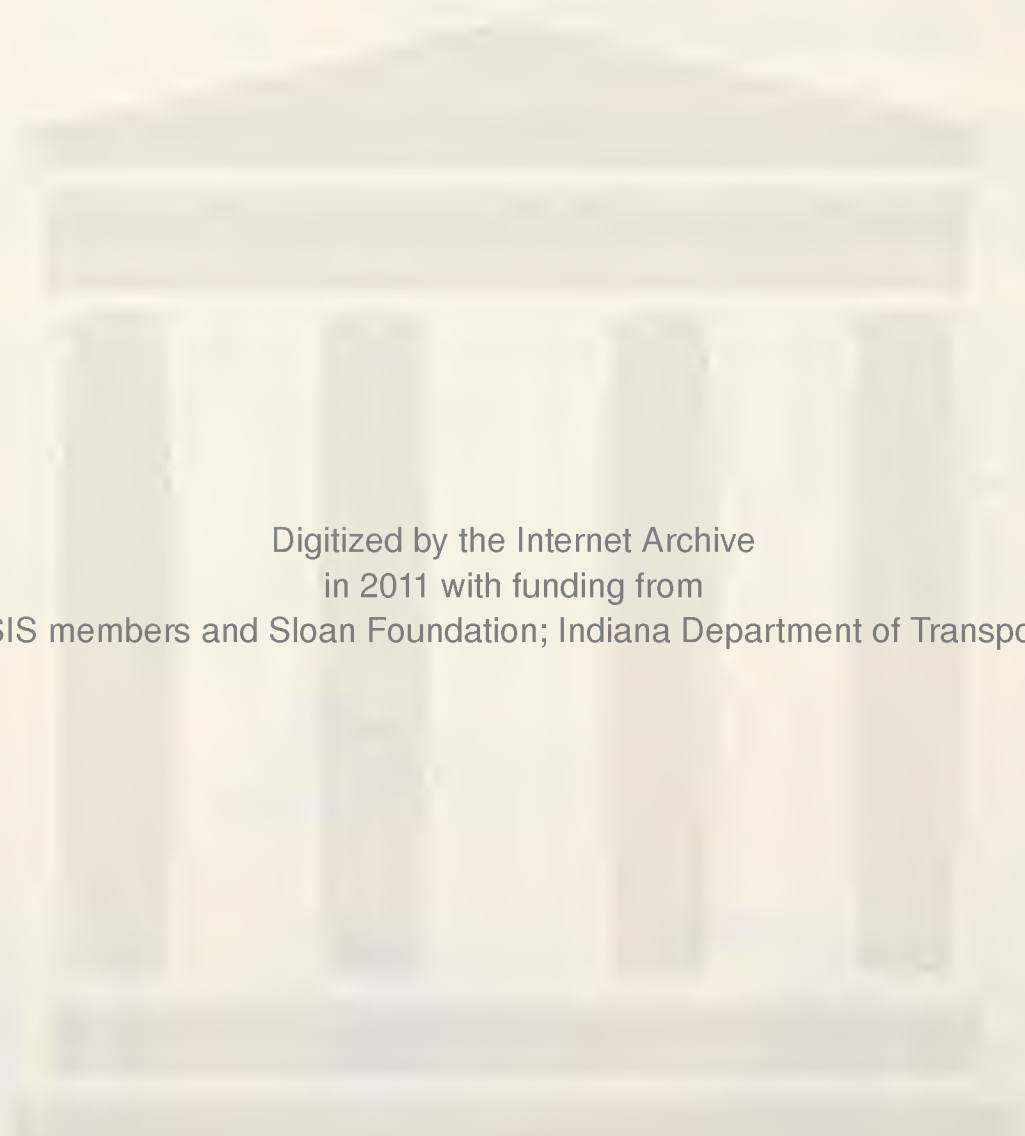
C. W. Lovell  
Research Engineer

CWL:ms

cc: A. G. Altschaeffl  
W. L. Dolch  
R. L. Eskew  
G. D. Gibson  
W. H. Goetz  
M. J. Gutzwiller  
G. K. Hallock

D. E. Hancher  
K. R. Hoover  
J. F. McLaughlin  
R. D. Miles  
P. L. Owens  
G. T. Satterly

C. F. Scholer  
K. C. Sinha  
C. A. Venable  
H. P. Wehrenberg  
L. E. Wood  
E. J. Yoder  
S. R. Yoder



Digitized by the Internet Archive  
in 2011 with funding from  
LYRASIS members and Sloan Foundation; Indiana Department of Transportation



Final Report  
GEOTECHNICAL DATA BANK FOR INDIANA

by

Y-K. T. Lo  
Graduate Instructor in Research

Joint Highway Research Project

Project No.: C-36-51T

File No.: 1-5-20

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project  
Engineering Experiment Station  
Purdue University

in cooperation with the  
Indiana State Highway Commission  
and the

U.S. Department of Transportation  
Federal Highway Administration

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Purdue University  
West Lafayette, Indiana  
May 14, 1980



## TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA/IN/JHRP-80/7	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle GEOTECHNICAL DATA BANK FOR INDIANA		5. Report Date May 14, 1980	
		6. Performing Organization Code	
7. Author(s) Y-K. T. Lo		8. Performing Organization Report No. JHRP-80-7	
9. Performing Organization Name and Address Joint Highway Research Project Civil Engineering Building Purdue University West Lafayette, Indiana 47907		10. Work Unit No.	
		11. Contract or Grant No. HPR-PL-1(15) Part I	
12. Sponsoring Agency Name and Address Indiana State Highway Commission State Office Building 100 North Senate Avenue Indianapolis, Indiana 46204		13. Type of Report and Period Covered  Final Report	
		14. Sponsoring Agency Code	
15. Supplementary Notes Prepared in cooperation with the U.S. Dept. of Transportation, Federal Highway Administration for the study "A Computerized Information Storage and Retrieval System for Soils of Indiana".			
16. Abstract <p>Nearly all readily accessible geotechnical data have been stored, bringing the data base to nearly 10000 sets. Several types of statistical operations were undertaken on the data, including median model characterizations of frequency distributions and regression analysis to produce soil parameter prediction.</p> <p>The most difficult aspect of statistical operation on the data is selection of the appropriate data population or grouping. Data have been treated in the following groups: state-wide, physiographic section, parent material area, AASHTO classification, and certain combinations thereof. The proper grouping seems to vary with the frequency distribution being examined or the prediction being attempted.</p> <p>Selected usage of the data bank is recommended via specific examples. Further research is recommended and is planned by the cooperating agencies.</p>			
17. Key Words Soils data, computer, data storage, data retrieval, statistics, median model, prediction regression analysis.		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 539	22. Price



## ACKNOWLEDGEMENTS

The research described was carried out under the sponsorship of the Joint Highway Research Project of Purdue University, the Indiana State Highway Commission, and the Federal Highway Administration. Their support is gratefully acknowledged.

The author wishes to express sincere appreciation to his major professor, Dr. C. W. Lovell for his encouragement, guidance, and friendship.

The author thanks Professor R. D. Miles for his assistance throughout the project, including valuable editorial aid, Dr. G. P. McCabe for his statistical advice, and Dr. T. R. West for serving in the examination committee and giving valuable opinions.

Mr. W. J. Sisiliano and Mr. R. W. Rahn of Indiana State Highway Commission gave valuable opinions throughout this work and aided in the development of examples. Mr. C. Andrews provided locational data for those samples where they were not obvious from the report.



Special thanks are due to those involved in the collection of project data, without whom this work would not be possible.

To Dr. H. T. David, Professor of statistics at Iowa State University, who guided the author's formal study in statistics and provided valuable guidance in data analysis during his stay at Iowa State, thanks are expressed.





## TABLE OF CONTENTS

	Page
LIST OF TABLES.....	viii
LIST OF FIGURES.....	xxiii
NOMENCLATURE.....	xxvii
HIGHLIGHT SUMMARY .....	xxxiii
CHAPTER 1 - INTRODUCTION.....	1
CHAPTER 2 - PREVIOUS INVESTIGATIONS.....	5
2.1       The Concept of a Geotechnical Data Bank	5
2.2       Empirical Relationships in the Practice of Geotechnical Engineering.....	7
2.2.1.    Consolidation Parameters.....	7
2.2.1. a   Compression Index ( $C_c$ ).....	8
2.2.1. b   Compression Ratio ( $C_r$ ).....	13
2.2.1. c   Preconsolidation Pressure ( $p_c$ ).....	14
2.2.2.    Compaction Parameters.....	17
2.2.2. a   Optimum Moisture Content (OMC) and Maximum Dry Density ( $\rho_{dmax}$ ).....	17
2.2.2. b   California Bearing Ratio (CBR).....	18
2.2.3.    Strength Parameters.....	24
2.2.3. a   Unconfined Compressive Strength ( $q_u$ )..	26
2.2.3. b   Undrained Shear Strength ( $c_u$ ).....	29
2.2.3. c   Standard Penetration Resistance (N) and Strength Angle ( $\phi'$ ).....	31
2.3.       Discussion.....	33



### CHAPTER 3 - THE DEVELOPMENT OF THE GEOTECHNICAL DATA BANK FOR THE STATE OF INDIANA

3.1.	General.....	35
3.2.	Source and Structure of Data.....	35
3.3.	Methods of Analysis.....	41
3.3.1. a	Median Model.....	41
3.3.1. b	The Application of the Median Model.....	48
3.3.2. a	Regression Model.....	65
3.3.2. b	The Application of the Repression Model..	78

### CHAPTER 4 - DISCUSSIONS OF RESULTS

4.1.	General.....	93
4.2.	The Median Model and Soil Variability....	93
4.2. a	Topographic Characteristics Versus Physiographic Regions.....	95
4.2. b	Remolded Soil Characteristics Versus Physiographic Regions and AASHTO Classifications.....	96
4.2. c	Soil Characteristics Versus Soil Associations.....	97
4.2. d	Soil Series as a Grouping Unit.....	98
4.3.	The Regression Model and Correlations....	99
4.3. a	Compaction Parameters.....	99
4.3. b	Consolidation Parameters.....	100
4.3. c	Strength Parameters.....	102

### CHAPTER 5 - APPLICATIONS

5.1.	Topographic Features.....	115
5.2.	Shallow Foundations.....	115
5.2. a	Bearing Capacity.....	115
5.2. b	Settlement.....	116
5.3.	Pile Foundations.....	117
5.4.	Cut Slope.....	118
5.5.	Excavations and Retaining Structures.....	118
5.6.	Compaction Requirements for Embankments..	118



	Page
5.7.	Pavements..... 119
5.8.	Example 1..... 119
5.8.1.	Project Identification..... 119
5.8.2.	General Scope of Procedures..... 120
5.8.3.	Determination of the Soil Types..... 120
5.8.4.	Determination of the Engineering and Strength Characteristics..... 121
5.8.5.	Recommendations for Preliminary Design... 128
5.8.5. a	Highway Embankment..... 128
5.8.5. b	Settlement of Embankment..... 128
5.8.5. c	Bridge Foundations..... 129
5.8.5. d	Retaining Walls..... 129
5.8.5. e	Ground Water..... 130
5.8.5. f	Piles, Corrosion..... 130
5.8.5. g	Others..... 130
5.8.6.	Recommendations for Subsurface Investigation Program..... 130
5.8.6. a	Recommendations using Data Bank..... 131
5.8.6. b	Other Recommendations..... 131
5.8.6. c	Conclusions..... 133
5.8.6. d	Summary..... 133
5.9.	Example 2..... 133
5.9. a	Project Identification..... 133
5.9. b	General Scope of Procedures..... 133
5.9. c	Determination of the Soil Types and Their Corresponding CBR Values..... 136
5.9. d	Recommendation of the Preliminary Design CBR Values..... 136
5.10.	Example 3..... 137
5.10.a	Procedures..... 137
5.10.b	Results..... 137

## CHAPTER 6 - SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1.	Summary..... 139
6.2.	Conclusions..... 140





	Page
6.3. Recommendations for Future Research.....	142
BIBLIOGRAPHY.....	144
APPENDICES *	
APPENDIX A: RESULTS.....	155
APPENDIX A-I: TOPOGRAPHIC CHARACTERISTICS.....	156
APPENDIX A-II: REMOLDED SOIL CHARACTERISTICS.....	163
APPENDIX A-III: STATISTICAL SOIL PROFILES.....	212
APPENDIX A-IV: REGRESSION EQUATIONS AND CORRELATIONS...	431
APPENDIX B: CODING SYSTEMS AND COMPUTER PROGRAMS.....	438
APPENDIX B-I: ERRATA TO THE USER'S MANUAL OF "THE DEVELOPMENT OF THE COMPUTERIZED GEO- TECHNICAL DATA BANK FOR THE STATE OF INDIANA" BY GARY D. GOLDBERG (35).....	439
APPENDIX B-II: DATA RETRIEVAL PROGRAMS.....	454
APPENDIX C: MULTIPLE COMPARISON TABLES.....	459

---

\*See page insert following page 154 for Appendices included in this copy of the Report.



## LIST OF TABLES

Table		Page
2-1	Summary of Published Regression Equations for Prediction of Compression Index, $C_c$ .....	9
2-2	Summary of Published Regression Equations for Prediction of Compression Ratio, $C'_r$ .....	15
2-3	Summary of Regression Equations for Prediction of Optimum Moisture Content (OMC) and Maximum Dry Density, $\rho_{dmax}$ .....	19
3-1	Distribution of Shrinkage Limits for the Soil Association Berks-Gilpin-Weikert with the Depth Less Than 5 Feet.....	43
3-2	Distribution of Shrinkage Limits for the Soil Association Cincinnati-Rossmoyne- Hickory with the Depth Less Than 4 Feet.....	46
3-3	Distribution of Natural Moisture Contents for the Soil Association Huntington- Wheeling-Markham with the Depth between 5 and 10 Feet.....	47
3-4	A Summary of Results from Table 3-1 to 3-3....	49
3-5	Distribution of AASHTO Units for Soil Association Wellston-Zanesville-Berks.....	53
3-6	Distribution of Counties for Soil Association Wellston-Zanesville-Berks.....	53



	Page
3-7      Distribution of Physiographic Regions for Soil Association Wellston-Zanesville-Berks.....	54
3-8      Distribution of Ground Elevations Which Have Corresponding WATER FINAL Records for Soil Association Wellston-Zanesville-Berks.....	54
3-9      Distribution of WATER FINAL for Soil Association Wellston-Zanesville-Berks.....	54
3-10     Distribution of Ground Water Elevation (= Elevation of Ground Surface - WATER FINAL) for Soil Association Wellston- Zanesville-Berks.....	55
3-11     Distribution of DEPTH T of A-4 Soils for Soil Association Wellston-Zanesville-Berks....	55
3-12     Distribution of DEPTH B of A-4 Soils for Soil Association Wellston-Zanesville-Berks....	57
3-13     Distribution of DEPTH T of A-6 Soils for Soil Association Wellston-Zanesville-Berks....	57
3-14     Distribution of DEPTH B of A-6 Soils for Soil Association Wellston-Zanesville-Berks....	58
3-15     Distribution of DEPTH T of A-7-6 Soils for Soil Association Wellston-Zanesville-Berks....	59
3-16     Distribution of DEPTH B of A-7-6 Soils for Soil Association Wellston-Zanesville-Berks....	60
3-17     AASHTO Units vs. DEPTH T for Soil Association Wellston-Zanesville-Berks.....	62



	Page
3-18 Statistical Soil Profile for Soil Association Wellston-Zanesville-Berks.....	66
3-19 Correlation Table for Maximum Dry Density ( $\rho_{dmax}$ ) versus Some Index Properties.....	79
4-1 One-way Classification of Soil Variable ( $\hat{\theta}_{ij}$ ).....	94
5-1 Engineering and Strength Characteristics of East Chicago Soils.....	126
5-2 The Quantity Estimates of the Recommended Subsurface Investigation Program.....	134

#### Appendix Table

A-I-1 Ground Elevation (ft.), Indiana.....	157
A-I-2 Water Depth with Relation to Ground Elevation (ft.), Indiana.....	159
A-I-3 Ground Water Elevation (ft.), Indiana.....	161
A-II-1 Remolded Soil Characteristics within the Tipton Till Plain.....	165
A-II-2 Remolded Soil Characteristics within the Tipton Till Plain.....	166
A-II-3 Remolded Soil Characteristics within the Tipton Till Plain.....	167





	Page
A-II-4 Remolded Soil Characteristics within the Tipton Till Plain.....	168
A-II-5 Remolded Soil Characteristics within the Dearborn Upland.....	170
A-II-6 Remolded Soil Characteristics within the Dearborn Upland.....	171
A-II-7 Remolded Soil Characteristics within the Dearborn Upland.....	172
A-II-8 Remolded Soil Characteristics within the Muscatatuck Regional Slope.....	174
A-II-9 Remolded Soil Characteristics within the Muscatatuck Regional Slope.....	175
A-II-10 Remolded Soil Characteristics within the Scottsburg Lowland.....	177
A-II-11 Remolded Soil Characteristics within the Scottsburg Lowland.....	178
A-II-12 Remolded Soil Characteristics within the Scottsburg Lowland.....	179
A-II-13 Remolded Soil Characteristics within the Norman Upland.....	181
A-II-14 Remolded Soil Characteristics within the Norman Upland.....	182
A-II-15 Remolded Soil Characteristics within the Norman Upland.....	183
A-II-16 Remolded Soil Characteristics within the Mitchell Plain.....	185



## Page

A-II-17	Remolded Soil Characteristics within the Mitchell Plain.....	186
A-II-18	Remolded Soil Characteristics within the Mitchell Plain.....	187
A-II-19	Remolded Soil Characteristics within the Crawford Upland.....	189
A-II-20	Remolded Soil Characteristics within the Crawford Upland.....	190
A-II-21	Remolded Soil Characteristics within the Crawford Upland.....	191
A-II-22	Remolded Soil Characteristics within the Wabash Lowland.....	193
A-II-23	Remolded Soil Characteristics within the Wabash Lowland.....	194.
A-II-24	Remolded Soil Characteristics within the Wabash Lowland.....	195
A-II-25	Remolded Soil Characteristics within the Calumet Lacustrine Section.....	197
A-II-26	Remolded Soil Characteristics within the Calumet Lacustrine Section.....	198
A-II-27	Remolded Soil Characteristics within the Calumet Lacustrine Section.....	199
A-II-28	Remolded Soil Characteristics within the Valparaiso Moraine.....	201
A-II-29	Remolded Soil Characteristics within the Valparaiso Moraine.....	202



A-II-30	Remolded Soil Characteristics within the Valparaiso Moraine.....	203
A-II-31	Remolded Soil Characteristics within the Kankakee Lacustrine Section.....	205
A-II-32	Remolded Soil Characteristics within the Kankakee Lacustrine Section.....	206
A-II-33	Remolded Soil Characteristics within the Kankakee Lacustrine Section.....	207
A-II-34	Remolded Soil Characteristics within the Steuben Morainal Section.....	209
A-II-35	Remolded Soil Characteristics within the Steuben Morainal Section.....	210
A-II-36	Remolded Soil Characteristics within the Steuben Morainal Section.....	211

#### List of Statistical Soil Profiles:

Tipton Till Plain .....	213
A-III-1	Statistical Soil Profile for the Soil
Association Ockley-Westland.....	214
A-III-2	Statistical Soil Profile for the Soil
Association Ockley-Fox.....	218
A-III-3	Statistical Soil Profile for the Soil
Association Blount-Pewama .....	223





	Page
A-III-4    Statistical Soil Profile for the Soil Association Crosby-Brookston.....	228
A-III-5    Statistical Soil Profile for the Soil Association Fincastle-Ragsdale-Brookston.....	235
A-III-6    Statistical Soil Profile for the Soil Association Miami-Crosby.....	240
A-III-7    Statistical Soil Profile for the Soil Association Miami-Hennepin.....	245
A-III-8    Statistical Soil Profile for the Soil Association Morley-Blount.....	252
A-III-9    Statistical Soil Profile for the Soil Association Odell-Chalmers.....	257
A-III-10   Statistical Soil Profile for the Soil Association Sidell-Parr.....	261
 Dearborn Upland .....	 266
A-III-11   Statistical Soil Profile for the Soil Association Huntington-Wheeling-Markland.....	267
A-III-12   Statistical Soil Profile for the Soil Association Avonburg-Clermont.....	272
A-III-13   Statistical Soil Profile for the Soil Association Cincinnati-Rossmoyne-Hickory.....	276



## Page

Scottsburg Lowland .....	280
A-III-14 Statistical Soil Profile for the Soil Association Fox-Nineveh-Ockley.....	281
A-III-15 Statistical Soil Profile for the Soil Association Berks-Gilpin-Weikert.....	285
Norman Upland .....	289
A-III-16 Statistical Soil Profile for the Soil Association Crider-Hagerstown-Frederick.....	290
Mitchell Plain .....	295
A-III-17 Statistical Soil Profile for the Soil Association Crider-Hagerstown-Frederick.....	296
A-III-18 Statistical Soil Profile for the Soil Association Crider-Frederick.....	303
A-III-19 Statistical Soil Profile for the Soil Association Corydon-Weikert-Berks.....	308



	Page
Crawford Upland .....	314
A-III-20 Statistical Soil Profile for the Soil Association Haymond-Wakeland.....	315
A-III-21 Statistical Soil Profile for the Soil Association Wellston-Zanesville-Berks.....	325
Wabash Lowland .....	335
A-III-22 Statistical Soil Profile for the Soil Association Wakeland-Stendal-Haymond- Bartle.....	336
A-III-23 Statistical Soil Profile for the Soil Association McGary.....	343
A-III-24 Statistical Soil Profile for the Soil Association Vincennes-Zipp-Ross.....	348
A-III-25 Statistical Soil Profile for the Soil Association Cincinnati-Hickory.....	353
A-III-26 Statistical Soil Profile for the Soil Association Zanesville-Wellston.....	358
A-III-27 Statistical Soil Profile for the Soil Association Alford.....	365
A-III-28 Statistical Soil Profile for the Soil Association Bloomfield-Princeton-Ayrshire.....	371



A-III-29	Statistical Soil Profile for the Soil Association Hosmer.....	377
	Calumet Lacustrine Section.....	384
A-III-30	Statistical Soil Profile for the Soil Association Alida-Del Rey-Whitaker.....	385
	Valparaiso Moraine .....	391
A-III-31	Statistical Soil Profile for the Soil Association Elliott-Markham-Pewano.....	392
	Kankakee Lacustrine Section.....	399
A-III-32	Statistical Soil Profile for the Soil Association Plainfield-Brems-Morocco.....	400
A-III-33	Statistical Soil Profile for the Soil Association Rensselaer-Darroch.....	402





	Page
Steuben Morainal Section .....	405
A-III-34 Statistical Soil Profile for the Soil Association Oshtemo-Fox.....	406
A-III-35 Statistical Soil Profile for the Soil Association Miami-Riddles-Crosier (I).....	412
A-III-36 Statistical Soil Profile for the Soil Association Miami-Riddles-Crosier (II).....	415
A-III-37 Statistical Soil Profile for the Soil Association Miami-Owosso-Riddles.....	420
A-III-38 Statistical Soil Profile for the Soil Association Morley-Blount.....	425
C-1 The Multiple Comparison Tests of Ground Elevation (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	473
C-2 The Two Sample Comparison Tests of Ground Elevations (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	474
C-3 The Two Sample Comparison Tests of Ground Elevation (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	475
C-4 The Two Sample Comparison Tests of Ground Elevations (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	476
C-5 The Two Sample Comparison Tests of Ground Elevation (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	477



C-6	The Two Sample Comparison Tests of Ground Elevation (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	478
C-7	The Two Sample Comparison Tests of Ground Elevation (GRDSUR, ft.) by Physiographic Regions (PHYSIO)*.....	479
C-8	The Multiple Comparison Tests of Ground Water Elevation (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	480
C-9	The Two Sample Tests of Ground Water Elevations (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	481
C-10	The Two Sample Tests of Ground Water Elevations (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	482
C-11	The Two Sample Tests of Ground Water Elevations (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	483
C-12	The Two Sample Tests of Ground Water Elevations (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	484
C-13	The Two Sample Tests of Ground Water Elevations (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	485
C-14	The Two Sample Tests of Ground Water Elevations (GWT, ft.) by Physiographic Regions (PHYSIO)*.....	486



C-15	The Multiple Comparison Tests of Maximum Dry Density (MAXDD or $\rho_{dmax}$ , pcf) by Physiographic Regions (PHYSIO)*.....	487
C-16	The Multiple Comparison Tests of Maximum Dry Density (MAXDD or $\rho_{dmax}$ , pcf) by AASHTO Classifications**.....	488
C-17	The Two Sample Comparison Tests of Maximum Dry Density (MAXDD or $\rho_{dmax}$ , pcf) by AASHTO Classifications**.....	489
C-18	The Two Sample Comparison Tests of Maximum Dry Density (MAXDD or $\rho_{dmax}$ , pcf) by AASHTO Classifications**.....	490
C-19	The Two Sample Comparison Tests of Maximum Dry Density (MAXDD or $\rho_{dmax}$ , pcf) by AASHTO Classifications**.....	492
C-20	The Multiple Comparison Tests of Optimum Moisture Content (OPTIMC or OMC, %) by Physiographic Regions*.....	494
C-21	The Multiple Comparison Tests of Optimum Moisture Content (OPTIMC or OMC, %) by AASHTO Classifications**.....	495
C-22	The Two Sample Comparison Tests of Optimum Moisture Content (OPTIMC or OMC, %) by AASHTO Classifications**.....	496
C-23	The Two Sample Comparison Tests of Optimum Moisture Content (OPTIMC of OMC, %) by AASHTO Classifications**.....	498



C-24	The Two Sample Comparison Tests of Optimum Moisture Content (OPTIMC or OMC, %) by AASHTO Classifications**.....	499
C-25	The Multiple Comparison Tests of Natural Dry Density (NATDD or $\rho_d$ , pcf) by Soil Associations (ASSOC)***.....	500
C-26	The Multiple Comparison Tests of Unconfined Compressive Strength (QUSTR or $q_u$ , TsF) by Soil Associations (ASSOC)***.....	504
C-27	The Multiple Comparison Tests of Natural Dry Density (NATDD or $\rho_d$ , pcf) by Soil Associations (ASSOC)***.....	508
C-28	The Multiple Comparison Tests of Unconfined Compressive Strength (QUSTR or $q_u$ , TsF) by Soil Associations (ASSOC)***.....	509
C-29	The Multiple Comparison Tests of Natural Dry Density (NATDD or $\rho_d$ , pcf) by Soil Associations (ASSOC)***.....	510
C-30	The Multiple Comparison Tests of Natural Dry Density (NATDD or $\rho_d$ , pcf) by Soil Associations (ASSOC)***.....	511
C-31	The Multiple Comparison Tests of Natural Dry Density (NATDD or $\rho_d$ , pcf) by Soil Series (SERIES)****.....	512
C-32	The Multiple Comparison Tests of Unconfined Compressive Strength (QUSTR or $q_u$ , TsF) by Soil Series (SERIES)****.....	524





C-33	The Two Sample Comparison Tests of Unconfined Compressive Strength (QUSTR or $q_u$ , Tsf) by Soil Series (SERIES)****.....	536
C-34	The Two Sample Comparison Tests of Natural Dry Density (NATDD or $\rho_d$ , pcf) by Soil Series (SERIES)****.....	537
C-35	The Two Sample Comparison Tests of Unconfined Compressive Strength (QUSTR or $q_u$ , Tsf) by Soil Series (SERIES)****.....	538



## LIST OF FIGURES

Figure		Page
2-1	Approximate Interrelationships of Soil Classifications and Soaked California Bearing Ratio (after Liu (54)).....	25
3-1	Soil Data Distribution Map, Indiana Geotechnical Data Bank.....	36
3-2	Data Input Form (DIF).....	38
3-3	Construction of a Soil Layer System.....	61
3-4	Maximum Dry Density vs. Plastic Limit, Indiana Soils.....	80
3-5	Maximum Dry Density vs. Liquid Limit, Indiana Soils.....	81
3-6	Measured Maximum Dry Density vs. Predicted Maximum Dry Density, Indiana Soils.....	91
3-7	Measured Compression Index vs. Predicted Compression Index, Indiana Soils.....	92
4-1	Preconsolidation Pressure vs. Liquidity Index, Indiana Soils.....	101



4-2	Comparison of Measured and Predicted Values of Preconsolidated Pressure (Data from Indiana Geotechnical Data Bank).....	103
4-3	Strength Angle vs. Liquid Limit, Indiana Soils.....	104
4-4	Strength Angle vs. Plasticity Index, Indiana Soils.....	105
4-5	Cohesion vs. Liquid Limit, Indiana Soils.....	106
4-6	Cohesion vs. Plasticity Index, Indiana Soils.....	107
4-7	Unconfined Compressive Strength vs. Natural Dry Density, Indiana Soils.....	108
4-8	Unconfined Compressive Strength vs. Natural Dry Density, Indiana Soils.....	109
4-9	Unconfined Compressive Strength vs. Liquidity Index, Indiana Soils.....	110
4-10	Unconfined Compressive Strength vs. SPT, Indiana Soils.....	113
4-11	Strength Angle vs. SPT, Indiana Soils.....	114
5-1	Moisture Content vs. Depth, East Chicago (Data from Indiana Geotechnical Data Bank)....	122
5-2	Natural Dry Density vs. Depth, East Chicago (Data from Indiana Geotechnical Data Bank)....	123



5-3	SPT vs. Depth, East Chicago (Data from Indiana Geotechnical Data Bank).....	124
5-4	Unconfined Compressive Strength vs. Depth, East Chicago (Data from Indiana Geotechnical Data Bank).....	125

## Appendix Figure

A-II-1	Distribution of AASHTO Classification Units within the Tipton Till Plain.....	164
A-II-2	Distribution of AASHTO Classification Units within the Dearborn Upland.....	169
A-II-3	Distribution of AASHTO Classification Units within the Muscatatuck Regional Slope.....	173
A-II-4	Distribution of AASHTO Classification Units within the Scottsburg Lowland.....	176
A-II-5	Distribution of AASHTO Classification Units within the Norman Upland.....	180
A-II-6	Distribution of AASHTO Classification Units within the Mitchell Plain.....	184
A-II-7	Distribution of AASHTO Classification Units within the Crawford Upland.....	188
A-II-8	Distribution of AASHTO Classification Units within the Wabash Lowland.....	192
A-II-9	Distribution of AASHTO Classification Units within the Calumet Lacustrine Section.....	196





A-II-10	Distribution of AASHTO Classification Units within the Valparaiso Moraine.....	200
A-II-11	Distribution of AASHTO Classification Units within the Kankakee Lacustrine Section.....	204
A-II-12	Distribution of AASHTO Classification Units within the Steuben Morainal Section.....	208



## NOMENCLATURE

English

<u>Symbol</u>	<u>Represents</u>
AASH(T)O	American Association of State Highway (and Transportation) Officials
ANOV	Analysis of variance
Bedrk	Depth to bedrock
CBR	California bearing ratio
CBR SO1	Soaked California bearing ratio at 100% of standard Proctor maximum dry density
CBR SO2	Soaked California bearing ratio at 95% of standard Proctor maximum dry density
$C_c$	Compression index
CDC	Control Data Corporation
CI	Consistency index
C.I.	Confidence interval
$C_r$	Recompression index
$C'_r$	Compression ratio
$c_u$	Undrained shear strength
$c_v$	Coefficient of consolidation



<u>Symbol</u>	<u>Represents</u>
c	Vane shear strength or cohesion as specified
c'	Effective cohesion
D	Specimen diameter
DIF	Data input form
D <sub>r</sub>	Relative dry density
d.f.	Degree of freedom
E	Modulus of elasticity
El	Elevation
e	void ratio
e <sub>o</sub>	initial void ratio
FM	Full model
G	Specific gravity
GI	Group index
IR	Interquartile range
I <sub>p</sub>	Plasticity index
ISHC	Indiana State Highway Commission
LI	Liquidity index
MSE	Mean square due to error
MSR	Mean square due to regression
m	a factor (= 0.127 E/c)



<u>Symbol</u>	<u>Represents</u>
m.s.	Mean square
N	SPT values (resistance)
$N_q$	Terzaghi bearing capacity factor
n	Number of observations (samples)
OMC	Optimum moisture content
P	Failure load under plate load test
PCA	Portland Cement Association
PFRA	The Prairie Farm Rehabilitation Administration
PUCC	Purdue University Computing Center
$p_c$	Preconsolidation pressure
$p_o$	Overburden pressure
$p'$	Mean principal effective stress (= $1/3 (\sigma_1' + 2\sigma_3')$ )
q	Deviator stress (= $\sigma_1 - \sigma_3$ )
$q_u$	Unconfined compressive strength
$\hat{q}_c$	Estimated compressive strength
R	Correlation coefficient
R	Multiple correlation coefficient
$R^2$	Square of (multiple) correlation coefficient
RM	Reduced model





<u>Symbol</u>	<u>Represents</u>
S	Degree of saturation
Si	Initial degree of saturation
SL	Shrinkage limit
SPSS	Statistical Package for the Social Sciences
SPT	Standard penetration test
s	Suction (potential capillary rise)
sand	Percentage of sand by weight
silt	Percentage of silt by weight
s.d.(S.D.)	Standard deviation
s.d. of est.	Standard deviation of estimate
s.s.	Sum of square
Type	Type of strength tests
$t_{.25}$	25th percentile
$t_{.50}$	sample median
$t_{.75}$	75th percentile
UNIF	Unified soil classification system
U.S.D.A.	United States Department of Agriculture
w	Water content or moisture content
$w_L$	Liquid limit
$w_n$	Natural moisture content
$w_p$	Plastic limit



<u>Symbol</u>	<u>Represents</u>
<u>Greek</u>	
$\Theta$ $\theta$ theta	
$\theta$	Population median
$\hat{\theta}$	Estimated population median (sample median)
$\mu$ mu	
$\mu$	population mean
$\hat{\mu}$	sample mean
$\rho$ rho	
$\rho_d$	Natural dry density
$\rho_{dmax}$	Standard Proctor maximum dry density
$\Sigma$ $\sigma$ sigma	
$\sigma_1', \sigma_3'$	Effective stresses
$\sigma_1'$	Vertical effective stress
$\sigma_3'$	Cell pressure less background pore pressure in fully drained triaxial test
$\sigma_{ff}$	Normal stress on the failure plane at failure
$\tau$ tau	
$\tau_{ff}$	Shear stress on the failure plane at failure



<u>Symbol</u>	<u>Represents</u>
$\Phi$ $\phi$ phi	
$\phi$	Strength angle
$\phi'$	effective stress strength angle



## HIGHLIGHT SUMMARY

Efforts to extract the geotechnical information contained in the subsurface investigation reports held in the files of the Indiana State Highway Commission (ISHC) have extended over several years. A total of 9442 sets of data have been stored through early 1978 to mid 1979.

Both conventional and nonparametric statistical methods were employed. The data were grouped by using physiographic region or engineering soil classification, or soil association, or a combination of them. One-way classification, two-way classification, and factorial experiment layouts were used to examine the distributions of the data. Regression analysis was used to investigate the functional relationships between design parameters and index properties.

Topographic features were found to be significantly different among physiographic regions. Remolded soil characteristics were also evaluated and contrasted between





physiographic regions but also within AASHTO classifications. Statistical soil profiles were developed showing the relations of soil characteristics versus depth for soil association within a physiographic region. Finally, soil design parameters were estimated by using index properties.



## CHAPTER 1 INTRODUCTION

The need for knowledge of pedological and engineering soils information for use in planning, site selection, design, construction and maintenance of transportation facilities is recognized by civil engineers. Data are necessarily limited in quantity and quality due to economic and time constraints. While large amounts of detailed soil data are often available from work performed on adjacent or nearby projects, these data are usually not readily accessible for use or their existence is unknown.

The accumulation of laboratory and field test data by the Indiana State Highway Commission for characterizing the engineering properties of Indiana soils is extensive. The information is retained in the form of subsurface investigation reports, prepared by or for consulting design firms and governmental agencies for use in routine soil investigations. In its present voluminous form, the majority of these data are not very useful. The need exists to make this information more accessible for the engineer interested in detailed information of a site and the engineer interested in soil characteristics over a larger region.



In July 1977, a proposal was submitted to develop and test a computerized information storage and retrieval system for soils in Indiana. The specific objectives of the research effort were to:

1. Identify the sources of individual soil sample data and devise a system for soils data collection and codification.
2. Develop computer information storage programs which are flexible enough to meet the data needs of both the Joint Highway Research Project and the Indiana State Highway Commission.
3. Collect and store data for approximately 7500 soil samples from existing subsurface investigation reports.
4. Statistically correlate significant soil properties with standard soil types and develop interrelationships between selected soil properties.

To achieve these objectives the project was divided into two phases. In the first phase, the input format and computer storage and retrieval systems were developed, and 2508 soil data sets were placed in the bank. Gary Goldberg prepared an Interim Report, JHRP-78-6, dated June 1, 1978, entitled "The Development of the Computerized Geotechnical Data Bank for the State of Indiana" (35). As a part of this phase, a complete instructional user's manual was prepared. Limited statistical analysis of



stored data to January 1978 indicated that grouping of soils by physiographic regions and parent material areas appeared to be justified. The twelve parent material designations used to correlate the engineering soil test data were identified relative to the pedological soil series on an appropriate map. The engineering soils-parent material map for Indiana (62) was not used because of its very small scale and an inability to establish the locations of specific boring sites on it.

Phase II of the research is the subject of this report. It involved the storage of 6934 additional data sets, for a total of 9442 data sets as of December 1979. These data sets were from roadway soil boring reports and from those boring reports for bridge and culvert sites that contained laboratory test data. The bridge and culvert site boring reports were included as an extension and expansion of the project scope in March 1979. It was intended as a part of this extension that standard penetration test data at bridge and culvert sites would also be stored. The gathering of the data for the bridge and culvert sites extended until June 1979. At this time it was discontinued, since the costs of adding the standard penetration data were deemed excessive.

The purposes of Phase II of the study were:

1. to complete the storage of readily available engineering soil test data;





2. to show how to manage the data bank;
3. to evaluate the information stored, and develop correlations and quantitative values for planning and preliminary design by using statistical methods.

Both conventional and nonparametric statistical methods were employed, as discussed in Section 3.3. However, the nonparametric statistical methods appear to fit and explain the varieties of soil characteristics in a superior way. This is further explained in Section 3.3.1.a.

The data were grouped by using physiographic regions, engineering soil classifications, soil associations, or a combination of them. One-way classification, two-way classification, and factorial experiment layouts were used to examine the distributions of the data. Regression analysis was used to investigate the functional relationships between design parameters and index properties. The results are shown in Appendices A-I through A-IV. The soils data were extremely variable in their characteristics, therefore choices of suitable groupings for study were the most difficult tasks in this investigation.

It is emphasized that the samples of soil and topographic characteristics are not uniformly distributed throughout a physiographic region. The soil data distribution map (Figure 3-1) should be consulted before attempting any interpretations of results.



## CHAPTER 2 PREVIOUS INVESTIGATIONS

### 2.1 The Concept of a Geotechnical Data Bank

Each year an enormous amount of geotechnical data generated by field and laboratory measurement are accumulated in the files of public agencies and private consultants throughout the United States and abroad. The data are contained in formal reports of subsurface investigations, and are checked and verified. These data are used in different functions, such as site selection, planning, design, construction and maintenance of new facilities, and reconstruction. Once the project is completed, further use of these data is limited because of their format.

In the State of South Dakota it was found (22) that many expensive soil testing programs were being conducted on new highway routes which were either close to or parallel to the old alignments on which extensive soil testing had been previously undertaken. In 1965 the South Dakota Department of Highways, in cooperation with the Soil Conservation Service, began a program to collect the accumulated geotechnical data from previous projects (22). The data were then stored in a computerized system



and analyzed by using statistical methods for the purposes of planning, location and preliminary design.

Spradling (109) developed a computerized data storage and retrieval system for the State of Kentucky. The Kentucky Department of Transportation devised an extensive coding system for data which were collected but not suitable for direct computer storage. Due to the completeness of this coding system, and its applicability to soil information in general, some of its details were adopted for the Indiana data bank (35).

Recently, the U.S. Department of Transportation published a state-of-the-art report which documented basic information on automatic data processing techniques used by eight states (Colorado, Georgia, Illinois, Louisiana, Minnesota, New York, Pennsylvania, and West Virginia) in managing test data for highway materials (24). Three basic data processing techniques, viz., batch information systems, on-line interactive information systems, and on-line interactive laboratory information systems, were in use.

The Prairie Farm Rehabilitation Administration (PFRA) of Canada collected data on the geotechnical properties of glacial till deposits, glacial lake deposits, and alluvial deposits from many of its previous projects in western Canada (82, 88). A number of empirical relationships between routine classification tests and consolidation



and strength characteristics were developed, as discussed in Section 2.2. The correlations were generated primarily to aid in evaluating information from new sites or to approximate geotechnical information for preliminary investigations.

Similar geotechnical data storage and retrieval systems were developed in the following countries: Sweden (58), Finland and Denmark (58), France (53), Rhodesia (41), Algeria (87), and South Africa (116).

## 2.2 Empirical Relationships in the Practice of Geotechnical Engineering

### 2.2.1. Consolidation Parameters:

Consolidation is defined as the process of densification of a soil under sustained loading caused by the expulsion of water from the pores. The laboratory oedometer test is used to determine the consolidation parameters, such as compression index ( $C_c$ ), recompression index ( $C_r$ ), and preconsolidation pressure ( $p_c$ ). Schmertmann (96) and others (52, 81) have recommended that initial void ratio ( $e_0$ ), compression index ( $C_c$ ), recompression index ( $C_r$ ), overburden pressure ( $p_0$ ), and preconsolidation pressure ( $p_c$ ) be used to construct the field virgin compression curves and to predict settlements of clay soils in





situ. For details of these constructions refer to (52, 81, 96).

There were also many attempts to predict the consolidation parameters from easy-to-measure soil indices, such as natural moisture content, liquid limit, and initial void ratio. A survey of these attempts is presented in the following sections.

2.2.1. a Compression Index ( $C_c$ ). The compression index ( $C_c$ ) has often been correlated with either the liquid limit ( $w_L$ ), the natural moisture content ( $w_n$ ), initial void ratio ( $e_o$ ), or a combination of these. Table 2-1 shows a summary of available regression equations, together with their region of applicability, for the prediction of  $C_c$ .

Wroth and Wood (123) proposed from experience that

$$w + A \ln c_u = \text{constant}$$

where  $w$  is moisture content,  $A$  is a constant, and  $c_u$  is the undrained shear strength. Also, from phase relations,  $V = 1 + e = 1 + Gw$  for saturated soils, where  $V$  is the specific volume, i.e., volume of total mass/volume of solids,  $e$  is the void ratio,  $G$  is the specific gravity,



Table 2-1. Summary of Published Regression Equations for Prediction of Compression Index,  $C_c$

Regression Equation	Region of Applicability	Reference
$C_c = 0.007 (w_L - 10)$	Remolded clays	Skempton, (103)
$C_c = 17.66 \times 10^{-5} w_n^2 + 5.93 \times 10^{-3} w_n$ $- 1.35 \times 10^{-1}$	Chicago clays	Peck et al (79)
$C_c = 1.15 (e_o - 0.35)$	All clays	Nishida (76)
$C_c = 0.30 (e_o - 0.27)$	Inorganic, cohesive soil; silt, some clays; silty clays; clay	Hough (43)
$C_c = 1.15 \times 10^{-2} w_n$	Organic soils	Moran, Proctor, Mueser & Rutledge, (68)
$C_c = 0.256 + 0.00106(w_L - 65)$ $+ 0.32(e_o - 0.84) \pm 0.063$	Heavy medium clays and silt in Brazil	Cozzolino (21)



Table 2-1. (continued)

Regression Equation	Region of Applicability	Reference
$C_c = 1.21 + 0.0072(w_L - 95) + 0.53(e_o - 1.87) \pm 0.32$	Soft clays and silts in Brazil	Cozzolino (21)
$C_c = 0.545 (e_o - 0.756)$	Peats in Japan	Watanabi et al (119)
$C_c = 0.016 (w_L - 60)$	Tokyo subsoils	Ohsaki (77)
$C_c = 0.086 w_n$	Peats in Japan	Maegnchi et al (60)
$C_c = 0.5(e_o - 1)$		
$C_c = 0.009 (w_L - 10)$	Normally consolidated clay	Terzaghi et al (113)
$C_c = 0.21 e_o + 0.0111 w_p - 0.17$	Indian soils	Sengupta et al (98)
$C_c = 0.21 e_o + 0.00314 w_L - 0.07$		



Table 2-1. (continued)

Regression Equation	Region of Applicability	Reference
$C_c = (0.0075 \sim 0.011) w_n$	Canadian soils	NRCC (73)
$C_c = (0.45 \sim 0.75) e_o$		
$C_c = 0.75 (e_o - 0.5)$	Soils of very low plasticity	Sowers (108)
$C_c = 0.01 w_n$	Chicago clays	Osterberg (78)
$C_c = 0.0087 w_L - 0.089$	Tamil Nadn clays in India	Raghavan et al (85)
$C_c = 0.39 e_o - 0.03$		
$C_c = 0.0093 w_n + 0.0142 \approx 0.9 \frac{w_n}{100}$	Tamil Nadn clays in India	Raghavan et al (85)
$C_c = 0.0047 w_L + 0.3473 e_o - 0.2312$		
$C_c = 0.0033 w_L + 0.0086 w_n - 0.1323$		





Table 2-1. (continued)

Regression Equation	Region of Applicability	Reference
$C_c + 0.40 (e_o - 0.25)$	Greek and U.S. soils	Azzouz et al (6)
$C_c = 0.01 (w_n - 5)$		
$C_c = 0.006 (w_L - 9)$		
$C_c = 0.37 (e_o + 0.003 w_L - 0.34)$		
$C_c = 0.40 (e_o + 0.001 w_n - 0.25)$		
$C_c = 0.009 w_n + 0.002 w_L - 0.10$		
$C_c = 0.37 (e_o + 0.003 w_L + 0.0004 w_n - 0.34)$		
$C_c = 0.013 (w_n - 7)$	Peaty soils in Hokkaido, Japan	Kogure et al (49)
$C_c = 0.00782 w_n^{1.07}$		
$C_c = 0.62 (e_o - 0.56)$		
$C_c = 0.370 e_o^{1.17}$		
$C_c = 0.621 \rho_d^{-1.37}$ ( $\rho_d$ : natural dry density in kg/cm <sup>3</sup> )		



and  $w$  is the moisture content expressed as a ratio for saturated soils. Adding the critical state relationships of Roscoe et al (91), and the assumption that the undrained shear strength at the plastic limit is 100 times that at the liquid limit, Skempton and Northey (104), Wroth and Wood went through a series of rational procedures and derived the following relationship between compression index ( $C_c$ ), plasticity index ( $I_p$ ), and specific gravity ( $G$ ) for remolded soils

$$C_c = \frac{1}{2} I_p G$$

For  $G = 2.7$ ,

$$C_c = 1.35 I_p$$

The design charts developed by the Canadian Prairie Farm Rehabilitation Administration (PFRA) also demonstrated the relationships of the compression index ( $C_c$ ) versus natural moisture content ( $w_n$ ), liquid limit ( $w_L$ ), and natural dry density ( $\rho_d$ ) for the soils in western Canada (82, 88).

2.2.1. b Compression Ratio ( $C'_r$ ). The compression ratio ( $C'_r$ ) is defined as  $C_c / (1 + e_o)$ . By examining this expression and Table 2.1 it can be concluded that: (1)



there is a correlation between compression index ( $C_c$ ) and compression ratio ( $C'_r$ ) and; (2) compression ratio is correlated with liquid limit, moisture content, initial void ratio, or a combination of these.

Rutledge (93) and Fadum (29) showed that as the natural moisture content increases, the compression ratio increases linearly for normally consolidated clays. Table 2-2 gives a summary of available regression equations, together with their geographic region of applicability, for the prediction of  $C'_r$ .

2.2.1. c Preconsolidation Pressure ( $p_c$ ). By definition, the preconsolidation pressure ( $p_c$ ) is the greatest effective pressure the soil has carried in the past. Preconsolidation may be caused by a variety of factors (52) including:

- (1) Removal of overburden
- (2) Fluctuations in the groundwater table
- (3) Cold-welding of mineral points between particles
- (4) Exchange of cations
- (5) Precipitation of cementing agents
- (6) Geochemical processes caused by weathering
- (7) Delayed compression
- (8) Tectonic forces due to movements in the earth's crust.



Table 2-2. Summary of Published Regression Equations for Prediction  
of Compression Ratio,  $C'_r$

Regression Equation	Region of Applicability	Reference
$C'_r = 0.208 e_o + 0.0083$	Chicago clays	Peck et al (79)
$C'_r = 0.156 e_o + 0.0107$	All clays	Elnaggar et al (27)
$C'_r = 0.14 (e_o + 0.007)$	Greek and U.S. soils	Azzouz et al (6)
$C'_r = 0.003 (w_n + 7)$		
$C'_r = 0.002 (w_L + 9)$		
$C'_r = 0.126 (e_o + 0.003 w_L - 0.06)$		
$C'_r = 0.142 (e_o - 0.0009 w_n + 0.006)$		
$C'_r = 0.003 w_n + 0.0006 w_L + 0.004$		
$C'_r = 0.135 (e_o + 0.01 w_L - 0.002 w_n - 0.06)$		





Kogure and Ohira (49) proposed the following relationships:

$$p_c = 43.9 w_n^{-0.913}; (R = \text{correlation coefficient} = 0.821)$$

and

$$p_c = 1.65 e_o^{-0.988}; (R = \text{correlation coefficient} = -0.810)$$

where  $w_n$  is the natural moisture content, and  $e_o$  the initial void ratio for peat soil and underlying clays of the Ishikari area of Hokkaido, Japan.

The Canadian PFRA (82,88) described a relationship between liquidity index (LI) and  $\log p_c$ , where as LI increases,  $\log p_c$  decreases linearly for the soils in western Canada. An examination of this relationship shows that the data pattern appears scattered; the standard error of estimate is accordingly large. The U.S. Navy (118) proposed a similar relationship between LI and  $\log p_c$ , with consideration of the dependence of preconsolidation pressure on the soil sensitivity. Bjerrum (11) developed a relationship between the (pre-consolidation pressure/overburden pressure) ratio and plastic index for late glacial and post glacial clays.



### 2.2.2. Compaction Parameters

Compaction is defined as the densification of a soil by means of mechanical manipulation at constant water content, and is measured quantitatively in terms of dry density of the soil.

Proctor (84) demonstrated that there is a definite relationship between density and moisture content. The characteristic peak in the Proctor curve is known as the maximum dry density and its corresponding moisture content as optimum moisture content (OMC). Compaction specifications are written in terms of percentage compaction, a common requirement being 95% to 100%. Percentage compaction is based on the ratio of the field density to the maximum dry density obtained with a specified compaction effort, such as the standard AASHTO or modified AASHTO test, in the laboratory. Thus, the maximum dry density is often regarded as the total objective of the compaction operation.

2.2.2. a Optimum Moisture Content (OMC) and Maximum Dry Density ( $\rho_{d \text{ max}}$ ). The plasticity characteristics of a cohesive soil reflect its ability to hold water. The less tightly absorbed or highly oriented the water, the greater the freedom of movement for solids in the compaction process (50). Therefore, as the liquid limit, or plastic limit, or plastic index increases, OMC increases and



$\rho_d \text{ max}$  decreases. Furthermore, as the optimum moisture content increases, the maximum dry density decreases.

These relationships were investigated and verified by Woods and Litehiser (122), the U.S. Navy (118), Narayana Murty (69), and PFRA (82, 88) for a variety of soils.

Table 2-3 shows a summary of regression equations, together with their geographical regions of applicability, for the prediction of OMC and  $\rho_d \text{ max}$ .

2.2.2. b California Bearing Ratio (CBR). The CBR test is used to provide a low deformation measure of strength of compacted subgrade soil and is used with empirical curves to design asphalt pavement structures. In the literature, CBR values have been predicted by means of index properties, strength characteristics, and soil classification units.

(1) Evaluation of CBR values in terms of index properties. A relationship between the CBR and the group index (GI) was suggested by the Asphalt Institute (4), and later by Gawith and Perrin (31). Both CBR values were measured at 90% modified AASHTO maximum dry density. As the group index increases, the CBR decreases. Gawith and Perrin (31) also suggested the following equations for the prediction of the CBR values for Australian soils:

$$\begin{aligned} \text{Log CBR} = & 1.886 - 0.0143D - 0.00045A + 0.00515 \frac{B}{A} \\ & - 0.0000456 \left(\frac{B}{A}\right)^2 - 0.0037E \end{aligned}$$



Table 2-3. Summary of Regression Equations for Prediction of Optimum Moisture Content (OMC) and Maximum Dry Density ( $\rho_d \text{ max}$ )

Regression Equation	Region of Applicability	Reference
$\frac{K}{D_w} - \frac{1}{G} = \frac{x}{100} \left( -\frac{k}{D_w} \right) + \frac{1}{10\sqrt{m} + m(x - x_o)}$	all soils	Dewan (25)
<p>where K is density of water, <math>D_w</math> wet density of soil, G specific gravity, m constant, <math>x_o</math> moisture percentage, x optimum moisture content, and m obtained from</p>		
$\frac{K}{D_m} - \frac{1}{G} = \frac{x_o}{100} + \frac{1}{10\sqrt{m}}, \quad D_m \text{ maximum dry density.}$		
$OMC = \frac{w_L}{2} + \sqrt{\frac{w_L^2}{2} \left\{ 0.28 I_p + 0.495 x \right.}$ $\left. \sqrt{\frac{(w_L - I_p)^2}{2}} \right\}^2}$	Indian soils	Narayana Murty (71)





Table 2-3. (continued)

Regression Equation	Region of Applicability	Reference
$OMC = 1.0 w_p - 4.0$	Israel soils	Kassiff et al (48)
$OMC = 1/3 (w_L + 15)$	Indian soils	Ramiah et al (86)
$\rho_{d \max} = 2100 - 7.0 w_L$ (kg/cu. meter)	Israel soils	Kassiff et al (48)
$\rho_{d \max} = 2125 - 10 w_L$ (kg/cu. meter)	Indian soils	Ramiah et al (86)



where A is the percentage passing the No. 36 B.S. (British Standard) sieve, B is the percentage passing the No. 200 B.S. sieve, D the plasticity index and E the percentage passing the No. 7 B.S. sieve.

and

$$\text{CBR} = 4.5 + \frac{(20 - \text{GI})^2}{18}$$

where GI is the group index.

Stephenson, Karrah and Koplon (110) proposed the following equation to evaluate the CBR values for Alabama soils:

$$\text{Log CBR} = 2.334984 - 0.002425x_1 - 0.006920x_2$$

where  $x_1 = \%4 + \%10 + \%40 + \%60 + \%200$

(% of total sample passing each sieve size as indicated),

and  $x_2 = \% \text{ clay}$

(% of No. 10 fraction as determined by elutriation test).

Kassiff, et al (48) found that the CBR values increase with decrease in the difference between  $w_p$  and SL, and with increase in surcharge for Israel soils.

(2) Evaluation of CBR values in terms of strength characteristics. Robinson and Lewis (90) showed that a relationship existed between the CBR value and failure load P (in lb.) of a 3-inch square plate pushed into the ground. They proposed that:

$$\text{Log CBR} = 0.76837 \log P - 1.6442$$



Wiseman and Zeitlen (121) showed that:

$$\text{CBR} = mc$$

where  $c$  is the vane shear strength,

$m$  is a factor equal to  $0.127 E/c$

and  $E$  the modulus of elasticity of the soil.

The expression was taken from elastic theory, where the settlement of the CBR piston was calculated using a Poisson's ratio of 0.5. With the data from in-situ tests under existing pavements in Israel they suggested the following relationship:

$$\text{CBR} = 4.2c - 0.2$$

where CBR is expressed in percent, and the strength,  $c$ , in  $\text{kg/cm}^2$ , is measured from the vane test.

A rational approach, known as the suction method was proposed by Black (12), to estimate CBR values from plasticity and consistency indices. Consistency index (CI) =  $(w_L - w_n)/I_p$  where  $w_L$  is the liquid limit,  $w_n$  moisture content, and  $I_p$  plasticity index. This estimation was based on the assumption that at 0.1 inch penetration the soil is close to failure, and that the CBR is equal to  $sNq/10$ , where  $s$  is suction and  $Nq$  the Terzaghi's bearing capacity factor using an effective friction angle. From a knowledge of correlations between plasticity characteristics, suction, and effective angle of friction for saturated British clays, which follow the relationship



$I_p = 0.83 w_L - 14.2$ , and through a process of interpolation and extrapolation Black developed expressions for the CBR. An experimental relationship between unsaturated and saturated CBR values was also developed. Recently Black (13) assumed that  $CBR = c_u/23$ , where  $c_u$  is the undrained shear strength in kPa, and used a similar approach to attain the relationships between  $c_u$ , CBR, and plasticity index.

Livneh et al (55) and Greenstein et al (37) used a similar approach to predict CBR values using index characteristics for expansive clays and dune sands.

(3) Evaluation of CBR by Soil Classifications. The four principal variables affecting compacted CBR are soil density, moisture content, compaction effort and the texture and grading of the soil (1). The AASHTO and Unified soil classification systems are based on the textural characteristics and the plasticity characteristics of soil. Accordingly these two engineering soil classification systems have been widely used for the prediction of CBR values.

The U.S. Navy (118) correlated the values of typical characteristics, such as maximum dry density, optimum moisture content, and CBR of compacted materials against the Unified soil classification system. The American Hoist and Derrick Company (1) used the AASHTO classification system, along with qualitative descriptions of soil characteristics, to approximate CBR values.





Casagrande (17) suggested the interrelationship of the Unified system, Public Roads system (which later became AASHO and AASHTO systems), and CBR values. Similar interrelationships were also given by the Asphalt Institute (5) and the Portland Cement Association (83). Having summarized all these interrelationships and made a comparison of groups in the AASHTO and Unified soil classification system, Liu (54) proposed the approximate relative relationships of various groups of both systems to an empirical measure of the CBR values shown in Figure 2-1.

In Israel the AASHTO soil classification affords the best prediction for CBR values. The Canadians have also adopted the AASHTO soil classification, as quoted by Sharma (99), and used group index as well in the final correlation for predicting CBR values.

### 2.2.3. Strength Parameters

Shear strength is usually assumed to be made up of:

- (i) a frictional component which increases linearly with the normal stress on the failure plane, and
- (ii) a cohesive component which is constant.

The Mohr-Coulomb equation is ordinarily used to describe the strength and is merely the equation of a straight line:

$$\tau_{ff} = c + \sigma_{ff} \tan \phi$$

where  $\tau_{ff}$  is shear strength or shear stress on the failure



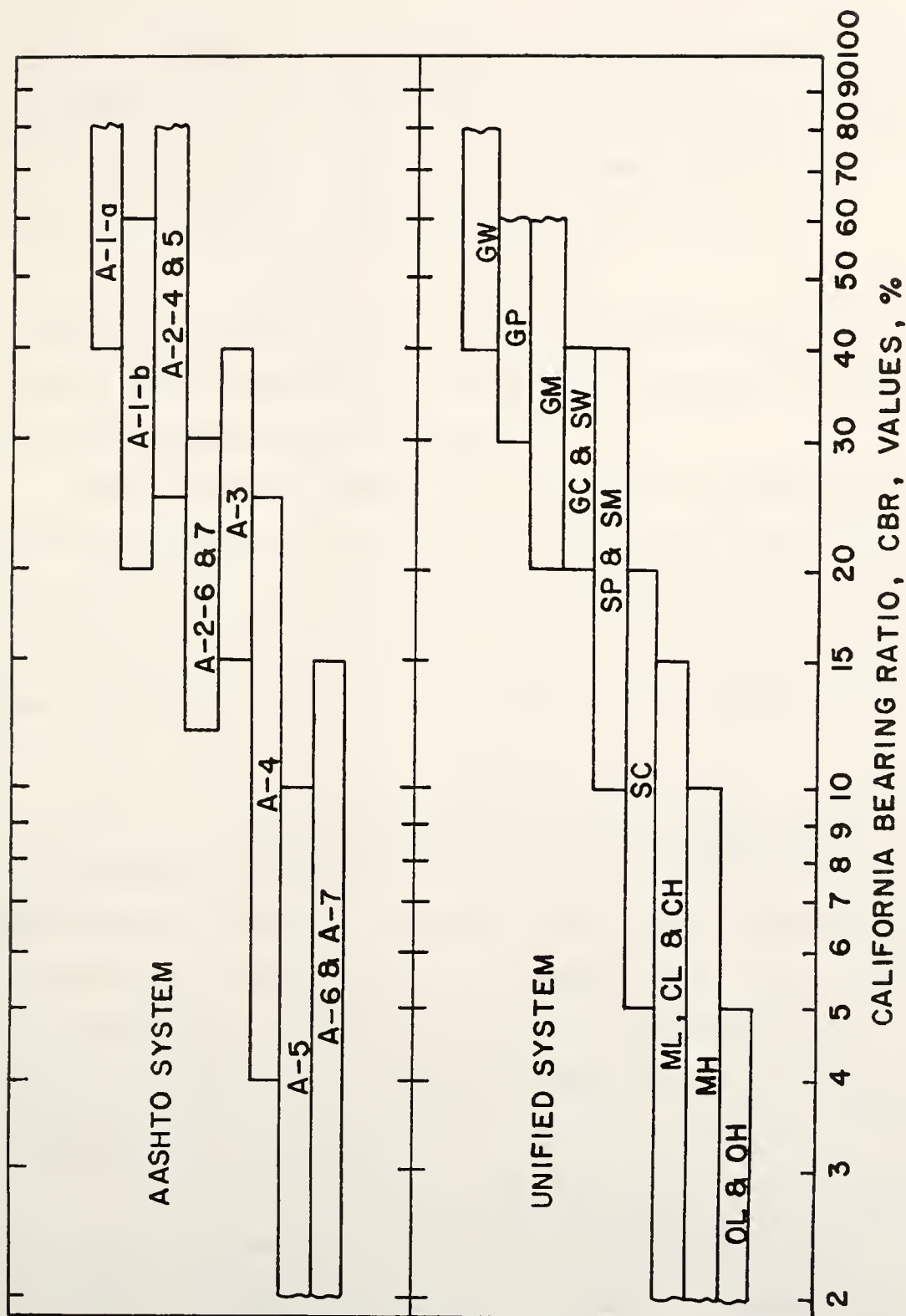


Figure 2-1. Approximate Interrelationships of Soil Classifications and Soaked California Bearing Ratio (after Liu (54))



plane at failure,  $c$  the cohesion or strength intercept,  $\phi_{ff}$  normal stress on the failure plane at failure, and  $\phi$  the strength angle. The equation is most rigorously written in terms of effective stress ( $\phi'$ ,  $c'$ ,  $\sigma'$ ):

$$\tau_{ff} = c' + \sigma'_{ff} \tan \phi'.$$

This relationship between shear strength and effective stress, first suggested by Terzaghi, is found to be valid for most engineering applications.

The three most common types of tests to determine the shear strength of soils are the direct shear test, the triaxial test, and the unconfined compression test. The unconfined compression test is actually a special case of the undrained triaxial test in which the confining pressure is zero.

2.2.3. a Unconfined Compressive Strength ( $q_u$ ). The Road Research Laboratory (89) described a curvilinear relationship between unconfined compressive strength ( $q_u$ ) and moisture content ( $w_n$ ) for a heavy clay in situ. This relationship was characterized by a large increase in unconfined compressive strength with decreasing moisture content below 30 percent.

Sherif and Burrous (100) investigated the effect of temperature on unconfined compressive strength of kaolinite. It was assumed that a change in temperature would cause a change in void ratio or a change in effective stress (or a combination of both) for a saturated compacted



clay. Thus, a change in temperature could cause a change in strength. Their test results showed that: (1) as the moisture content increases, the unconfined compressive strength decreases, and (2) at a given moisture content as the temperature increases, the unconfined compressive strength decreases. The temperature effects on the undrained strength for compacted soils were further investigated and verified by Hightner (39).

Sahasi (94) studied the correlation of unconfined compressive strength of soil with size of the test specimen. The Black Cotton soil (CH) at PUNE in India was used as the test material. His conclusions include the following: (1) there exists a unique relationship between the unconfined compressive strength and the moisture content, (2) the relation between unconfined compressive strength,  $q_u$  in  $\text{kg/cm}^2$ , moisture content,  $w_n$  in percentage, and specimen diameter,  $D$  in cm., may be expressed empirically as

$$\text{Log } q_u = 0.52 (0.502 - w_n - 0.163 \log D)$$

In his study on compacted clay of St. Croix, Indiana, Weitzel (120) found that at zero confining pressure the model for strength prediction dry-of-optimum is:

$$\hat{q}_c = -1784.8 + 3.1 \rho_d \sqrt{S_i} / w$$

where  $\hat{q}_c$  is the estimated compressive strength,  $\text{kN/m}^2$ ,





$\rho_d$  is the dry density,  $\text{kg/m}^3$ ,

$S_i$  is the initial degree of saturation, %,

and  $w$  is the water content, %.

The model for strength prediction wet-of-optimum is:

$$\log (\hat{q}_c) = 1.70/e_i$$

where  $e_i$  is the initial void ratio.

Peters and Lamb (82) presented an empirical relationship between liquidity index (LI) and unconfined compressive strength ( $q_u$ ) for the soils in western Canada. They showed that as the liquidity index decreases, the unconfined compressive strength increases.

Peck et al (80) suggested a relationship between the qualitative terms describing consistency, along with field identification of clays, and the quantitative values of the unconfined compressive strength. The National Research Council of Canada (74) also suggested a similar relationship for the rough estimate of the undrained shear strength (half of the unconfined compressive strength) for clay soils.

The unconfined compressive strength ( $q_u$ ) has been correlated with the standard penetration test (SPT), i.e., the number of blows (N-values) for one foot penetration. Some of these relationships are presented as follows. In his study on Tokyo subsoils, Ohsaki (77) found that the



relationship between  $q_u$  of clayey samples and their  $N$ -values was:

$$q_u(\text{kg/cm}^2) = 0.4 + \frac{N}{20}$$

However, an examination of the data shows much scatter. This expression, therefore, is qualitative rather than quantitative.

The U.S. Navy (118) recommended that for clayey silts, CL clays, or varved clays and silts,

$$q_u(\text{Tsf}) = 0.15N, \text{ and for CH clays}$$

$$q_u(\text{Tsf}) = 0.20N$$

Terzaghi and Peck (113) also suggested relations among consistency of clay described in qualitative terms,  $N$ -values, and unconfined compressive strength. Sanglerat (95) recommended the following expressions:

$$\text{for clay, } q_u(\text{Tsf}) = \frac{N}{4}$$

$$\text{for silty clay, } q_u(\text{Tsf}) = \frac{N}{5}$$

$$\text{and for silty, sandy soil, } q_u(\text{Tsf}) = \frac{N}{7.5}.$$

2.2.3. b Undrained Shear Strength ( $c_u$ ). The undrained shear strength ( $c_u$ ) under  $\phi = 0$  conditions is, in terms of unconfined compressive strength ( $q_u$ ),

$$c_u = \frac{1}{2} q_u$$

The undrained shear strength is also determined, in situ, by means of the vane shear test and other less common procedures.

Due to the fact that the undrained shear strength ( $c_u$ ) of normally consolidated clay soils is almost directly



proportional to the effective overburden pressure ( $p_o$ ), Skempton (105) suggested the following empirical relationship between the plastic index and the ratio  $c_u/p_o$ :

$$c_u/p_o = 0.11 + 0.0037 I_p$$

For saturated sand Bishop and Eldin (9) found that as the initial porosity ( $n$ ) increases, the ratio  $c_u/p_o$  decreases. And for a given initial porosity the higher the effective overburden pressure, the lower the ratio  $c_u/p_o$ .

Bjerrum and Simons (10) stated that the well-known Skempton-Bjerrum correlation of increasing ratio  $c_u/p_o$  with increasing plastic index applied only to normally consolidated marine clays of Norway. They also defined  $c_u/p_o$  as a function of the liquidity index (LI) for some Norwegian clays, to illustrate the existence of an unstable structure for quick clays with ratio  $c_u/p_o$  less than 0.15.

The U.S. Navy (118) suggested correlations of the unconfined undrained shear strength (or cohesion intercept in saturated state) with Unified classification. Of course, the unconfined undrained shear strength ( $c_u$ ) is zero for granular soils and is as high as 420 to 460 psf for silty clay or clayey silt soils.

In their study of the geotechnical characteristics of till deposits of the Quaternary Period in the Edmonton area, Alberta, Canada, May and Thomson (61) found a distinct trend for higher undrained shear strengths with lower moisture contents.



Using the assumption that the mean value of undrained shear strength is about  $1.70 \text{ kN/m}^2$  (from the results of Youssef et al (124)) for a variety of remolded Egyptian soils, Wroth and Wood (123) developed the following relationship

$$c_u = 170 \text{ EXP } (-4.6 \text{ LI}) \text{ kN/m}^2$$

2.2.3. c Standard Penetration Resistance (N) and Strength Angle ( $\phi'$ ) (1) Standard Penetration Resistance (N-value). The standard penetration resistance (N-value) varies with the relative density or relative consistency of soils, and is usually evaluated in terms of effective overburden pressure ( $p_o$ ) and relative dry density ( $D_r$ ) for cohesionless cases. Sanglerat (95) has documented many equations and charts from all over the world. One such equation was established by Bazaraa (7) in the form,

$$N = 20 D_r^2 (1 + 2P_o) \text{ for } p_o \leq 1.5 \text{ kips/ft}^2,$$

and  $N = 20 D_r^2 (3.25 + 0.5 P_o) \text{ for } p_o \geq 1.5 \text{ kips/ft}^2.$

The U.S. Navy (118) developed a correlation between N and relative dry density ( $D_r$ ) which included the effective overburden pressure ( $P_o$ ). Peck et al (80) related N and the relative density in qualitative terms for sands.

(2) Strength angle ( $\phi'$ ). Ohsaki (77) suggested the following relationship between  $\phi'$  and N for the sandy subsoils of Tokyo

$$\phi' = \sqrt{20 N + 15}.$$

However, an examination of the plot of  $\phi'$  versus N shows





that the data is scattered. For sand, Caquot and Kerisel (15) found a relationship

$$\tan \phi' = \frac{A}{e}$$

where  $e$  is void ratio, and  $A$  a constant assumed to be 0.55. Later (16), they proposed that under an overburden pressure of 1 bar ( $\approx 1$  ton/sq. ft.):

for silts:  $A = 0.4$

and for sands:  $0.45 < A < 0.55$ .

Meyerhof (65) proposed that the relationship between  $\phi'$  and  $D_r$  be expressed as

$$\phi' = 25 + 0.15 D_r,$$

for granular soil containing more than 5% fine sand and silt, and

$$\phi' = 30 + 0.15 D_r$$

for granular soil containing less than 5% fine sand and silt.  $D_r$  is relative dry density expressed as a percentage.

Bjerrum and Simons (10) presented the relationship between strength angle and plastic index for different clays. However, Terzaghi and Peck (113) questioned the general validity of this relationship, citing test data from Mexico City soils. Also, in their study of the strength characteristics of Kuttanad clays in India, Narain and Ramanathan (72) found that there was no definite correlation between strength angle and plastic index. However, the Canadian PFRA (82, 88) found that



as the liquid limit increased, the strength angle decreased for undisturbed clays in western Canada.

### 2.3. Discussion

Review of the large number of empirical relationships documented in the geotechnical engineering literature leads to two principal conclusions:

- (1) The values of soil parameters are expressed as means.
- (2) The functional relationships among soil characteristics are established with data from a certain region or pooled data from several regions. The regional effects on relationships are not commonly investigated or compared.

Jenny (45) gave a generalized equation form of soil-forming factors as:

$$s = f(cl, o, r, p, t....)$$

where s is soil, cl climate, o organisms, r topography, p parent material, and t time. These factors have imposed a random pattern of variation onto an overall trend of soil formation. The resulting soil characteristics can be regarded as being controlled by a random process. The random process of soil formation explains the great variation often encountered for a given soil parameter. Therefore, it seems more reasonable to define and use the median, rather than mean, for soil parameter values.



The regional effects on functional relationships among soil characteristics should be investigated by using the techniques of qualitative variables as regressors associated with analysis of variance. The details of these procedures are discussed and illustrated in Section 3.3.2.a.



## CHAPTER 3. THE DEVELOPMENT OF THE GEOTECHNICAL DATA BANK FOR THE STATE OF INDIANA

### 3.1. General

Goldberg (35) initiated the development of the Indiana geotechnical data bank. A total of 2508 data sets were collected and subjected to statistical analyses in an initial research phase. An additional 6934 data sets were added to the bank in the subsequent year and subjected to more detailed statistical analyses. As of January 1980, the Indiana geotechnical data bank contained 9442 data sets. The distribution of data sets throughout the State is shown in Figure 3-1.

In the following, the source of data, structure of data, and methods leading to the building of mathematical models are discussed. Results are given in Appendix A.

### 3.2. Source and Structure of Data

Both geotechnical and pedological soils information was collected. The geotechnical information was taken from the subsurface investigation reports of various geotechnical projects previously conducted in the State of Indiana, and the pedological soils information was from recent agriculture Soil Survey Manuals (105) and General Soil Maps





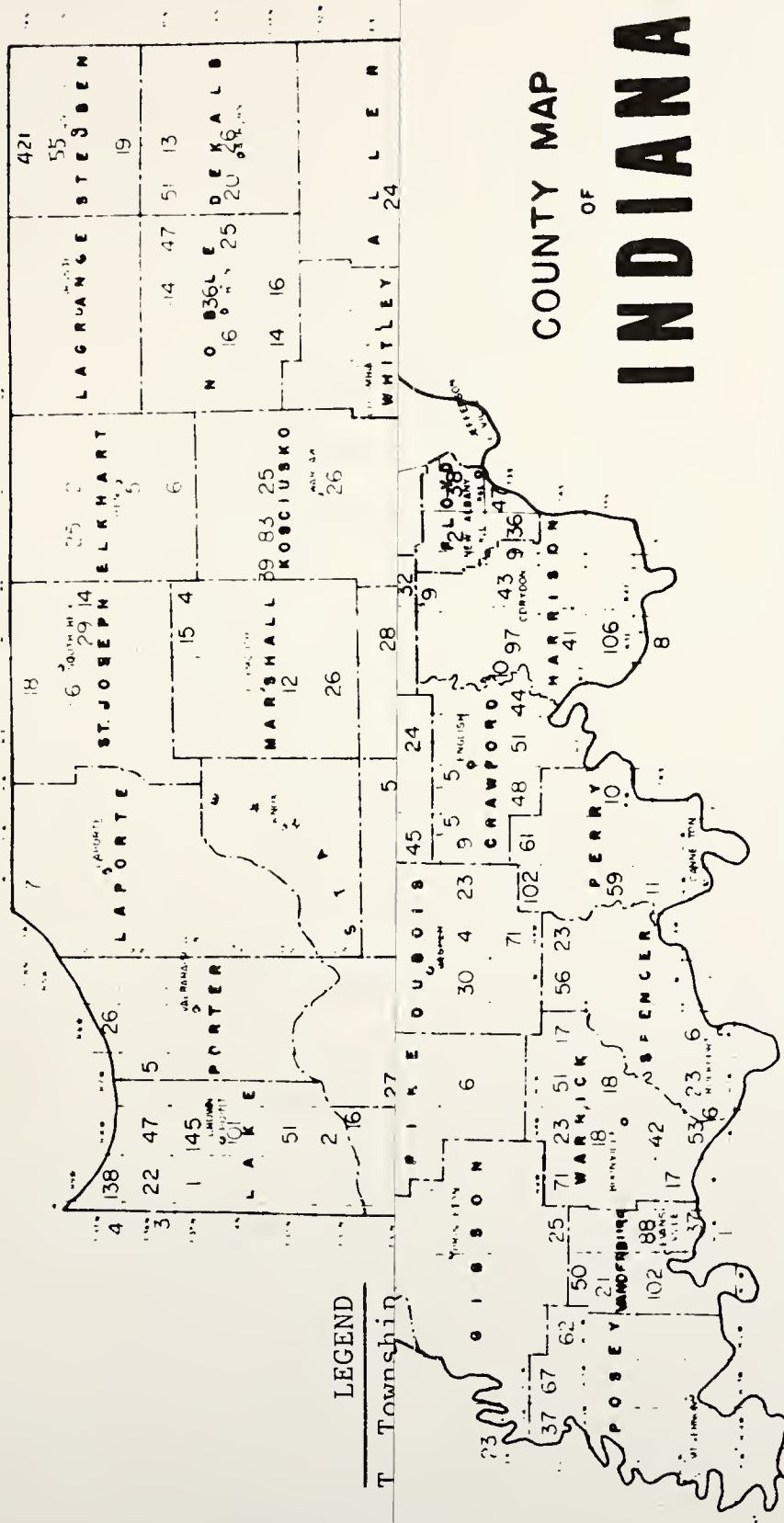


Figure 3-1. Soil Data Distribution Map, Indiana Geotechnical Data Bank



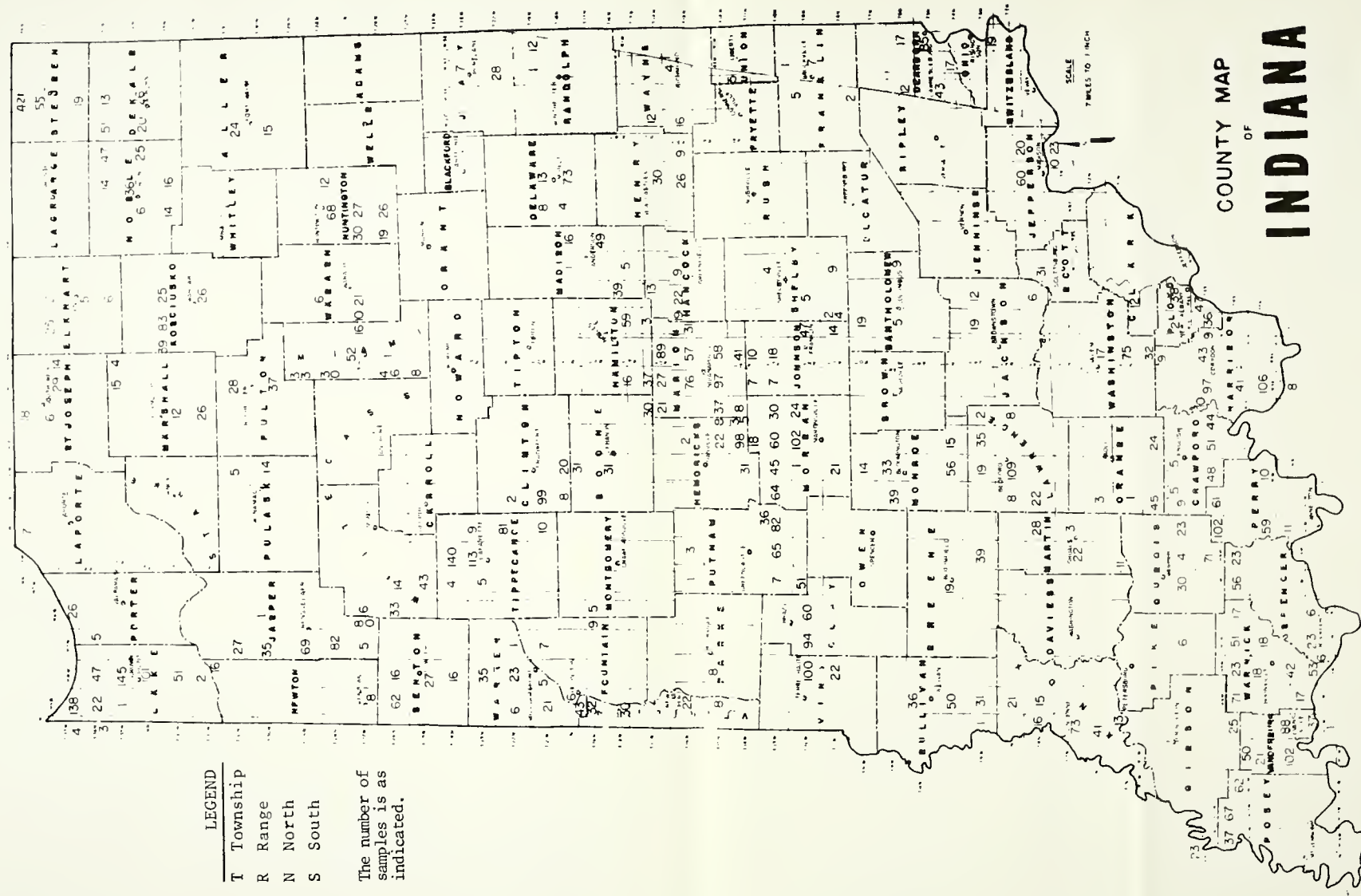


Figure 3-1. Soil Data Distribution Map, Indiana Geotechnical Data Bank



(32). For details refer to Goldberg (35).

The Data Input Form (DIF) of Figure 3-2 was developed to record the information. Not all items were available for all samples, but the listing included the following:

1. Project identification,
  - a. project number
  - b. contract number
  - c. road number
  - d. data collection agency
2. Sample location,
  - a. county
  - b. highway district
  - c. township
  - d. range
  - e. section
  - f. line number
  - g. station number
  - h. offset and the left or right direction from the center line.
3. Sample identification,
  - a. boring number
  - b. laboratory number
  - c. sampling procedure
4. Date the sample taken from the hole,
5. Physiographic region,



RECORDED BY: \_\_\_\_\_ DATE: \_\_\_\_\_

SEQNUM: \_\_\_\_\_ CHECKED BY: \_\_\_\_\_ DATE: \_\_\_\_\_

CARD	C O	HOLE NUMBER								S A M P L E N O.	NUMBER S U P	BORING NUMBER										SOIL ASSOC.		R E P E A T					
		1	2	3	4	5	6	7	8			9	10	64	65	66	67	68	69	70	71	72	73		74	75	76	77	78
1																													
2																													
CARD	ID NO.	DEPTH BOTTOM		SPT		P H Y S I C A L		SOIL SERIES																					
		1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	
2																													
CARD	ID NO.	E		3/4" SIEVE		1/2" SIEVE		3/8" SIEVE																					
		1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	
3																													
CARD	ID NO.	L		PI		SL		LOSS ON IGNIT %																					
		1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	
4																													
CARD	ID NO.	CBR SOAKED 100% MAX DD		CBR SOAKED 95% MAX DD		UNCONFINED COMPRESSIVE STRENGTH TSF		A A S H T O		U N I F																			
		1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	
5																													
CARD	ID NO.	TEST		P <sub>r</sub> TSF		C <sub>c</sub>		C <sub>r</sub>		C <sub>v</sub> FT <sup>2</sup> /MTH																			
		1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	
6																													

COMMENTS:

Figure 3-2 Data i





## DATA INPUT FORM

RECORDED BY: \_\_\_\_\_ DATE: \_\_\_\_\_

SEQNUM: \_\_\_\_\_

## COMPUTERIZED SOIL DATA FOR THE STATE OF INDIANA

CHECKED BY: \_\_\_\_\_ DATE: \_\_\_\_\_

CARD C O	HOLE NUMBER								SAMP NO	DIST	YEAR	MONTH	DAY	TOWN	DR	RANGE	OR	SPT	PROJECT NUMBER												CONTRACT NUMBER				ROAD NUMBER				BORING NUMBER				SOIL ASSOC.		REPEAT																																		
	PRE	NO.	PAREN	MILE	PRE	NO.	PRE	NO.											PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.	PRE	NO.		PRE	NO.	PRE	NO.																														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
2	ID NO.								STATION NUMBER								OFFSET				LINE NUMBER				LAB NUMBER				GROUND SURFACE ELEVATION				DEPTH TOP		DEPTH BOTTOM		SPT		PHYSIO		SOIL SERIES																																						
3	ID NO.								PARENT		HOR		SLOPE		OR		BEDRK SURVEY		BEDRK BORING LOG		WATER		WATER COMPLETION		WATER FINAL OR 24 HOURS		DRAIN		AREAL		FLOOD		FROST		STRAIN		PH		1-1/2" SIEVE		1" SIEVE		3/4" SIEVE		1/2" SIEVE		3/8" SIEVE																																
4	ID NO.								NO 4 SIEVE		NO 10 SIEVE		NO 40 SIEVE		NO 200 SIEVE		NO 270 SIEVE		% SAND		% SILT		% CLAY		% COLLOIDS		LL		PL		PI		SL		LOSS ON IGNIT %																																												
5	ID NO.								NATMC %		NATWD PCF		NATDD PCF		SPECIFIC GRAVITY		TEMP		OR		COLOR		COHESION		MAXDD PCF		MAXWD PCF		OPTIM MOIST CONTENT %		CBR UNSOAK 100% MAX OD		CBR UNSOAK 95% MAX OD		CBR SOAKED 100% MAX OD		CBR SOAKED 95% MAX OD		UNCONFINED COMPRESSIVE STRENGTH TSF		ASTM		UNIT																																				
6	ID NO.								FAILURE STRAIN %		TYPE		STRENGTH TESTS												CONSOLIDATION TEST																																																						
													STRENGTH TSF		STRAIN %		CONF. PRESS. PC		COHESION TSF		$\phi$		PORE PRESSURE PSI		$\sigma_c$ TSF		$e_o$		$e_i$		$S_o$ %		$S_i$ %		$P_o$ TSF		$P_i$ TSF		$C_c$		$C_r$		$C_v$ FT 2/ MTH																																				

COMMENTS:

Figure 3-2 Data input form (DIF)



6. Parent material from which the soil has been derived,
7. Ground surface elevation,
8. Depth from which the sample has been removed,
9. Depth to the bedrock,
10. Depth to groundwater,
11. Standard penetration resistance (SPT),
12. Pedological soils information,
  - a. soil association name
  - b. soil series name
  - c. horizon
  - d. slope (topographic) class
  - e. erosion class
  - f. natural soil drainage class
  - g. generalized permeability
  - h. generalized flooding potential
  - i. generalized frost heave susceptibility
  - j. generalized shrink - swell potential
  - k. generalized pH
13. Gradational characteristics based on standard sieve sizes and hydrometer analysis,
14. Atterberg limits,
15. Visual textual classifications,
16. Color based on moist conditions,
17. Organic content (loss on ignition),



18. In-situ moisture content,
19. In-situ dry and wet densities,
20. Specific gravity,
21. Compaction test results,
22. California bearing ratio (CBR),
23. Unconfined compressive strength and failure strain,
24. Strength data from triaxial and direct shear tests,
25. Consolidation test results.

Details of these listing and their corresponding coding systems are described in Goldberg (35). Computer programs to utilize the information from (13) to (16) to classify the samples by the AASHTO and Unified soil classification systems, to detect certain input errors, and to correlate a soil association with its most probable corresponding parent material are listed in Appendix B-II.

Those programs shown in Appendix B-II were written in the SPSS (Statistical Package for the Social Sciences) language available in the PUCC (Purdue University Computing Center) for CDC 6500 and 6600 systems. The SPSS language, like any other recent statistical language, is a "conversational" statistical analysis software. It also has features of data manipulation, data transformation, file definition, and file creation. For details refer to



Nie et al (75). This study has relied heavily upon this language. However, any computer language is merely an access to an optimum use of the computer hardware. Therefore, the logical sequences of a program are more important than the program itself. The logic of these programs, as listed in Appendix B-II, should be regarded as the guideline for the ready conversion to any other computer language. And the methods of analyses as described in the following are designed and written for adaptation to any language.

### 3.3. Methods of Analysis

Two types of prediction equations were used in this work: (1) median models, and (2) regression models.

#### 3.3.1.a. Median Model

To numerically define the variability of selected soil characteristics, the frequency distributions of these characteristics are examined and described. One way to describe the sample distribution is to use the conventional constant mean model (33), which is based upon the assumption of normality of the population distribution. This model is characterized by the mean and the standard deviation of the sample distribution. It is shown (51) that in most cases this model is also effective for moderately anormal distributions of a large sample. In the case of small samples, however, this model may not give





accurate approximations. The non-parametric or distribution-free methods are the preferred techniques of inference for non-normal population (Snedecor and Cochran (106)). These methods make a minimum of assumptions regarding the sample distribution and are generally appropriate for any form of the distribution. They are of high efficiency, relative to classical techniques, under the assumption of normality, and often of higher efficiency in other situations (51). For further details refer to Hollander et al (42) and Snedecor (106). In this study the samples distributions were sometimes normal, but frequently they were skewed or bimodal. The median model was used to characterize the sample distribution.

To estimate the median from a sample the observations are arranged in increasing order. When the sample values are arranged in this way, they are often called the 1st, 2nd, 3rd,...order statistics. In general, if  $n$  is odd, the median is the order statistic whose number is  $\frac{n+1}{2}$ . With  $n$  even, the median is defined as the average of the order statistics whose numbers are  $\frac{n}{2}$  and  $\frac{n+2}{2}$ . Like the mean, the median is a measure of the middle of a distribution. If the distribution is symmetrical about its mean, the mean and the median coincide. With highly skewed distributions the median is preferred, since it seems to better represent the concept of an average than the mean.



The calculations of the median and other non-parametric estimates from a large sample are illustrated in the following example with the data from the current Indiana geotechnical data bank. The example is for 43 observations of shrinkage limits for the soil association Berks-Gilpin-Weikert with the depth less than 5 feet. The observations are arranged in an increasing order and the proper intervals for the distribution are determined. The results are shown in Table 3-1. with the data from Indiana Geotechnical Data Bank.

Table 3-1. Distribution of Shrinkage Limits for the Soil Association Berks-Gilpin-Weikert with the Depth Less than 5 Feet

Shrinkage Limit (in %)	12.00- 15.00	15.00- 18.00	18.00- 21.00	21.00- 24.00	24.00- 27.00	27.00- 30.00	30.00- 33.00
Frequency	8	5	6	5	9	7	3
Cumulative Frequency	8	13	19	24	33	40	43

There are two modes in the distribution. The first mode is in the class from 12.00 to 15.00, and the second mode in the class from 24.00 to 27.00. This bimodal feature emphasizes the non-normality of the distribution.



Since the number of observation (n) is equal to 43, the sample median is the order statistic that is 22nd in the array. It is clear that the median is in the class of 21.00 - 24.00. The median is found by interpolation with the assumption of uniform distribution of observations in this class. The general equation is (106)

$$t_{.50} = x_L + \frac{gI}{f} \quad (3.1)$$

where  $x_L$  = value of x at lower limit of the class containing the median (21.00 in this case),

$g$  = order statistic number of the median minus cumulative frequency up to the upper limit of previous class ( $22 - 19 = 3$ ),

$I$  = class interval (3%),

$f$  = frequency in class containing the median (5).

This gives

$$\begin{aligned} t_{.50} = \text{median} &= 21.00 + \frac{(22.00 - 19.00)(3)}{5} \\ &= 22.80\% \end{aligned}$$

There is a simple method of calculating confidence intervals for the population median that is valid for any continuous distribution. Two of the order statistics serve as the upper and lower confidence interval limits. These are the order statistics whose numbers are, approximately (67)

$$\frac{n+1}{2} \pm \frac{z\sqrt{n}}{2}, \quad (3.2)$$



where  $z$  is the normal deviate corresponding to the desired confidence probability. Consider a 95% confidence probability,  $z \approx 2$ . In this example, these numbers are  $\frac{43 + 1}{2} \pm \frac{2\sqrt{43}}{2} = 15$  and 28. The 95% confidence intervals of the median are the shrinkage limits corresponding to the 15th and 28th order statistics. The actual shrinkage limits are found by adapting equation 3.1 for the median.

$$\begin{aligned}\text{For the 15th, shrinkage limit} &= 18 + (15 - 13) (3)/5 \\ &= 19.20\%\end{aligned}$$

$$\begin{aligned}\text{For the 28th, shrinkage limit} &= 24 + (28 - 24) (3)/9 \\ &= 25.33\%\end{aligned}$$

The population median is between 19.20% and 25.33% except for unusual values that occur about once in twenty trials.

In any continuous frequency distribution the  $p$ -th percentile is estimated by the order statistic whose number is  $(n + 1)p/100$ . For the 43 shrinkage limits, the 25th percentile is estimated by an order statistic whose number is  $i_{.25} = (43 + 1)25/100 = 11$ . Again by using equation 3.1, the shrinkage limit corresponding to the 25th order statistic is

$$t_{.25} = 15 + (11 - 8) (3)/5 = 16.80$$

In the same way, the number of the order statistic 75th percentile ( $t_{.75}$ ) is  $i_{.75} = 33$ ; and  $t_{.75} = 27.00$ . The interquartile range (IR) is defined as the difference between the 75th and the 25th percentiles. In this case,  $IR = 27.00 - 16.80 = 10.20$ . The IR is used as a measure of population





variability rather than standard deviation. The median, confidence interval of median, percentiles, and inter-quartile range are known as the non-parametric estimates of a sample distribution.

In this case, the mean of the distribution is 22.4 and the standard deviation is 5.632. The 95% confidence interval of the mean is, approximately,

$$22.4 \pm \frac{2(5.632)}{\sqrt{43}} = 20.68 \sim 24.12$$

Let us consider another example. Table 3-2 shows 35 observations of shrinkage limits for the soil association Cincinnati-Rossmoyne-Hickory with the depth less than 4 feet. By observation, the distribution is skewed.

Table 3-2. Distribution of Shrinkage Limits for the Soil Association Cincinnati-Rossmoyne-Hickory with the Depth Less than 4 Feet

Shrinkage Limit (%)	9.00- 12.00	12.00- 15.00	15.00- 18.00	18.00- 21.00	21.00- 24.00	24.00- 27.00	27.00- 30.00	30.00- 33.00
Frequency	2	6	13	8	3	1	1	1
Cumulative Frequency	2	8	21	29	32	33	34	35

The same procedures were employed to attain the non-parametric estimates of the sample distribution. The results are as follows.



The median,  $t_{.50}$ , is equal to 17.31 and the 95% confidence interval of the median is 15.92 - 19.13. The 25th and 75th percentiles are 15.23 and 20.25, respectively. Accordingly the IR = 5.02. The mean of this distribution is 18.37 and the standard deviation is 4.35. Therefore, the 95% confidence of the mean is, approximately,

$$18.37 \pm \frac{2(4.35)}{\sqrt{35}} = 16.90 \sim 19.84$$

For a final example, Table 3-3 shows 38 observations of natural moisture contents for the soil association Huntington-Wheeling-Markham with depths between 5 and 10 feet. This distribution is seen to be symmetrical.

Table 3-3. Distribution of Natural Moisture Contents for  
the Soil Association Huntington-Wheeling-  
Markham with Depths between 5 and 10 Feet

Natural Moisture Content (%)	15.00- 20.00	20.00- 25.00	25.00- 30.00	30.00- 35.00	35.00- 40.00
Frequency	1	7	20	8	2
Cumulative Frequency	1	8	28	36	38

The same procedures were employed. With the following results. The median is  $t_{.50} = 27.87$  and the 95% confidence interval of the median is 26.25 ~ 29.50. The 25th and 75th percentiles are 25.50 and 30.62, respectively.



The interquartile range (IR) = 5.12. The mean of this distribution is 27.77 and the standard deviation is 3.942. The 95% confidence interval of the mean is, approximately,

$$27.77 \pm \frac{2(3.942)}{\sqrt{38}} = 26.49 \sim 29.05$$

A summary of results obtained above is shown in Table 3-4. For a bimodal distribution, the interquartile range is greater than the standard deviation. This is expected due to the bimodal feature. In the case of a skewed distribution the mean is greater than the median because of the skewness. As for symmetrical distributions, since the interquartile range is greater than the standard deviation, the non-parametric method is less efficient than the classical techniques based upon the assumption of normality. However, the non-parametric method still yields reasonable results.

#### 3.3.1.b. Applications of the Median Model

The methods and procedures illustrated in Section 3.3.1.a. were used to describe the following characteristics:

- (1) topographic characteristics, viz., ground elevation, ground water elevation, and water depth with relation to ground elevation;
- (2) the relationship between the remolded soil characteristics and AASHTO classification within a physiographic region;



Table 3-4 A Summary of Results from Table 3-1 to Table 3-3

Source	Median ( $t_{.50}$ )	Mean ( $\hat{\mu}$ )	95% Confidence Interval of Median	95% Confidence Interval of Median	Interquartile Range (IR)	Standard Deviation (s.d.)
Table 3-1 (Bimodal Distribution)	22.80	22.40	19.20 ~ 25.33	20.68 ~ 24.12	10.20	5.63
Table 3-2 (Skewed Distribution)	17.31	18.37	15.92 ~ 19.13	16.90 ~ 19.83	5.02	4.35
Table 3-3 (Symmetrical Distribution)	27.87	27.77	26.25 ~ 29.50	26.49 ~ 29.05	5.12	3.94





and

(3) statistical soil profiles.

Each item is described as follows.

(1) Topographic characteristics:

The topographic characteristics were examined within physiographic regions. The ground water depth with relation to ground elevation was taken directly from the WATER FINAL as recorded on the DIF. The ground water elevation was defined as the difference between the ground elevation and the WATER FINAL.\* The procedures of analyses are outlined as follows under the assumption of a uniform distribution of the observations of these topographic characteristics throughout a physiographic region:

(a) Examine the sample distributions of these topographic characteristics for a desired physiographic region, and

(b) Apply the methods discussed in Section 3.3.1.a. to describe these sample distributions.

The results are shown in Appendix A-1.

(2) Relationships between the remolded soil characteristics and AASHTO classification within a physiographic region:

There exist many relationships between the soil classification, notably the AASHTO, and the characteristics of remolded soils, as discussed in Chapter 2. To develop these kinds of relationships on a regional basis, the following procedures were used.

\*WATER FINAL is defined as the final or 24 hour reading, whichever is reported in the drill logs, of the depth to water.



(a) Examine the distribution of AASHTO classification units within a given physiographic region (Appendix A-II) and select three or four most probable AASHTO classification units as the representative soil groups in the region.

(b) Examine the distributions of both visual texture and Unified classification units for each of the selected AASHTO classification units within the specific region. Select the most probable texture and Unified classification units as the representatives for soil identification and correlation.

(c) Apply the methods described in Section 3.3.1.a. to obtain the estimates of samples distributions of the following remolded soil characteristics: natural dry density ( $\rho_d$ ), specific gravity (G), shrinkage limit (SL), maximum dry density ( $\rho_{d \text{ max}}$ ), optimum moisture content (OMC), CBR value at 100% maximum dry density (CBR S01), and CBR value at 95% maximum dry density (CBR S02) for each AASHTO classification unit selected above. The results are shown in Appendix A-II.

### (3) Statistical soil profiles

An attempt has been made to generate soil profiles with statistical bases. As the statistical reasoning is based on the characteristics of an aggregate of sample observations, any discontinuity in the sample distribution must be eliminated. The statistical soil profiles were



generated according to the pedological soil associations, because these are the only soil grouping units which are reasonably large and grossly homogeneous. The procedures of generating a statistical soil profile for a given soil association are illustrated below.

Sufficient data were available for the soil association Wellston-Zanesville-Berks (Table 3-18). The items of general description and parent material in Table 3.18 were extracted from the "General Soils Maps" (32) and the "Key to Soils of Indiana" (30). Examinations of sample distributions of AASHTO classification units, counties, and physiographic regions for this soil association as shown in Tables 3-5 through 3-7 indicate that: (i) the A-4, A-6, and A-7-6 soils are more dominate than the others, and, therefore, are selected as the soil representatives for further layer divisions; and (ii) this kind of soil is mainly distributed in Crawford, Dubois, and Perry counties within the Crawford Upland physiographic regions.

Tables 3-8 to 3-10 show the distributions of topographic characteristics. Only those ground elevations which have corresponding WATER FINAL records were selected. The methods presented in Section 3.3.1.a. were employed to describe these samples distributions.

The 90th percentile of the DEPTH T (as recorded on the DIF) for each of the AASHTO units selected was



Table 3-5 Distribution of AASHTO Units for Soil  
Association Wellston-Zanesville-Berks

AASHTO Unit	Frequency	Relative Frequency (%)	Cumulative Frequency (%)
Unknown	7	2.6	2.6
A-1-B	2	0.8	3.4
A-2-4	19	7.1	10.5
A-2-6	3	1.1	11.7
A-4	73	27.4	39.1
A-6	77	28.9	68.0
A-7-5	12	4.5	72.6
A-7-6	73	27.4	100.0
Total	266	100.0	100.0

Table 3-6 Distribution of Counties for Soil  
Association Wellston-Zanesville-Berks

County	Frequency	Relative Frequency (%)	Cumulative Frequency (%)
Crawford	110	41.4	41.4
Dubois	67	25.2	66.6
Harrison	7	2.6	69.2
Orange	17	6.4	75.6
Perry	65	24.4	100.0
Total	266	100.0	100.0





Table 3-7 Distribution of Physiographic Regions for  
Soil Association Wellston-Zanesville-Berks

Physiographic Region	Frequency	Relative Frequency (%)	Cumulative Frequency (%)
Crawford Upland	252	94.7	94.7
Wabash Lowland	14	5.3	100.0
Total	266	100.0	100.0

Table 3-8 Distribution of Ground Elevations Which  
Have Corresponding WATER FINAL Records for  
Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	420- 470	470- 520	520- 570	570- 620	620- 670	670- 720	720- 770	770- 820
Frequency	20	8	11	6	8	2	1	3
Cumulative Frequency	20	28	39	45	53	55	56	59

Table 3-9 Distribution of WATER FINAL for Soil  
Association Wellston-Zanesville-Berks

Class Limit (ft.)	1- 5	5- 9	9- 13	13- 17	17- 21	21- 25	25- 29	29- 33	33- 37
Frequency	27	14	6	6	3	1	1	0	1



Table 3-9 (continued)

Cumulative Frequency	27	41	47	53	56	57	58	58	59
-------------------------	----	----	----	----	----	----	----	----	----

Table 3-10 Distribution of Ground Water Elevation  
(Elevation of Ground Surface - WATER FINAL)  
for Soil Association Wellston-Zanesville-  
Berks

Class Limit (ft.)	415- 465	465- 515	515- 565	565- 615	615- 665	665- 715	715- 765	765- 815
Frequency	19	10	11	5	8	2	1	3
Cumulative Frequency	19	29	40	45	53	55	56	59

Table 3-11 Distribution of DEPTH T of A-4 Soils for  
Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0- 1	1- 2	2- 3	3- 4	4- 5	5- 6	6- 7	9- 10	10- 11	11- 12	14- 15	15- 16	18- 19	20- 21	22- 23
Frequency	16	7	7	6	10	1	5	1	1	2	4	2	5	3	3
Cumulative Frequency	16	23	30	36	46	47	52	53	54	56	60	62	67	70	73



defined as its upper depth boundary in the profile, and the 90th percentile of DEPTH B (as recorded on the DIF), as its lower depth boundary. This was done to eliminate the extremes of DEPTH T and DEPTH B and, hopefully, to minimize the discontinuity of soil sample versus depth distributions. For example, as shown in Table 3-11, the number in order statistics corresponding to the 90th percentile of the distribution of DEPTH T of A-4 soils is  $(73 + 1)90\% = 67$ . Therefore, the 90th percentile is equal to 19 feet, which gives the upper depth boundary of A-4 soil. In the same manner the 90th percentile of the distribution of DEPTH B of A-4 soil is 21 feet, which defines the lower depth boundary (Table 3-12). Using the sample distributions described in Table 3-13 to 3-16, and applying the same procedures, the upper depth boundary is 15 feet and the lower depth boundary is 16 feet for A-6 soil. The upper depth boundary is 12 feet and the lower depth boundary 17.50 feet for A-7-6 soil. The results are shown in Figure 3-3(a). Table 3-17 shows the distribution of all available AASHTO units versus DEPTH T.

An examination of Figure 3-3(a) and Table 3-17 indicates that A-4, A-6, and A-7-6 soils are more dominate from the surface to a depth about 12 feet. The A-6 soil is more dominate from 12 feet to approximately 17 feet. Thus a three layer system is drawn for this soil profile.



Table 3-12 Distribution of DEPTH B of A-4 Soils for  
Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0- 1	1- 2	2- 3	3- 4	4- 5	5- 6	6- 7	7- 8	8- 9	9- 10
Frequency	1	9	5	10	4	6	12	2	1	2
Cumulative Frequency	1	10	15	25	29	35	47	49	50	52

Table 3-12 (continued)

Class Limit (ft.)	11- 12	12- 13	13- 14	15- 16	16- 17	17- 18	19- 20	20- 21	22- 23	24- 25
Frequency	1	1	2	1	3	2	4	1	3	3
Cumulative Frequency	53	54	56	57	60	62	66	67	70	73

Table 3-13 Distribution of DEPTH T of A-6 Soils for  
Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0 -1	1 -2	2 -3	3 -4	4 -5	5 -6	6 -7	7 -8	8 -9	9 -10	10 -11	11 -12
Frequency	25	5	6	4	9	2	2	1	2	3	1	4





Table 3-13 (continued)

---

Cumulative Frequency	25	30	36	40	48	51	53	54	56	59	60	64
-------------------------	----	----	----	----	----	----	----	----	----	----	----	----

---

Table 3-13 (continued)

---

Class Limit	12	13	14	15	16	18
(ft.)	-13	-14	-15	-16	-17	-19

---

Frequency	3	2	1	2	3	2
-----------	---	---	---	---	---	---

---

Cumulative Frequency	67	69	70	72	75	77
-------------------------	----	----	----	----	----	----

---

Table 3-14 Distribution of DEPTH B of A-6 Soils for  
Soil Association Wellston-Zanesville-Berks

---

Class Limit	0	1	2	3	4	5	6	7	8	9
(ft.)	-1	-2	-3	-4	-5	-6	-7	-8	-9	-10

---

Frequency	2	13	4	7	9	4	8	5	2	1
-----------	---	----	---	---	---	---	---	---	---	---

---

Cumulative Frequency	2	15	19	26	35	39	47	52	54	55
-------------------------	---	----	----	----	----	----	----	----	----	----

---



Table 3-14 (continued)

Class Limit	10	11	12	13	14	15	17	18	19	20
(ft.)	-11	-12	-13	-14	-15	-16	-18	-19	-20	-21
Frequency	4	1	1	4	3	2	2	3	1	1
Cumulative Frequency	59	60	61	65	68	70	72	75	76	77

Table 3-15 Distribution of DEPTH T of A-7-6 Soils for  
Soil Association Wellston-Zanesville-Berks

Class Limit	0	1	2	3	4	5	6	7	8	9
(ft.)	-1	-2	-3	-4	-5	-6	-7	-8	-9	-10
Frequency	13	7	7	7	8	5	6	1	4	4
Cumulative Frequency	13	20	27	34	42	47	53	54	58	62

Table 3-15 (continued)

Class Limit	10	11	13	15	18	19	20
(ft.)	-11	-12	-14	-16	-19	-20	-21
Frequency	2	3	1	2	1	1	1
Cumulative Frequency	64	67	68	70	71	72	73



Table 3-16 Distribution of DEPTH B of A-7-6 Soils for  
Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	1 -2	2 -3	3 -4	4 -5	5 -6	6 -7	7 -8	8 -9	9 -10	10 -11
Frequency	10	2	9	8	7	4	6	5	4	3
Cumulative Frequency	10	12	21	29	36	40	46	51	55	58

Table 3-16 (continued)

Class Limit (ft.)	11 -12	12 -13	13 -14	15 -16	17 -18	19 -20	22 -23	23 -24	49 -50
Frequency	3	2	2	1	3	1	1	1	1
Cumulative Frequency	61	63	65	66	69	70	71	72	73



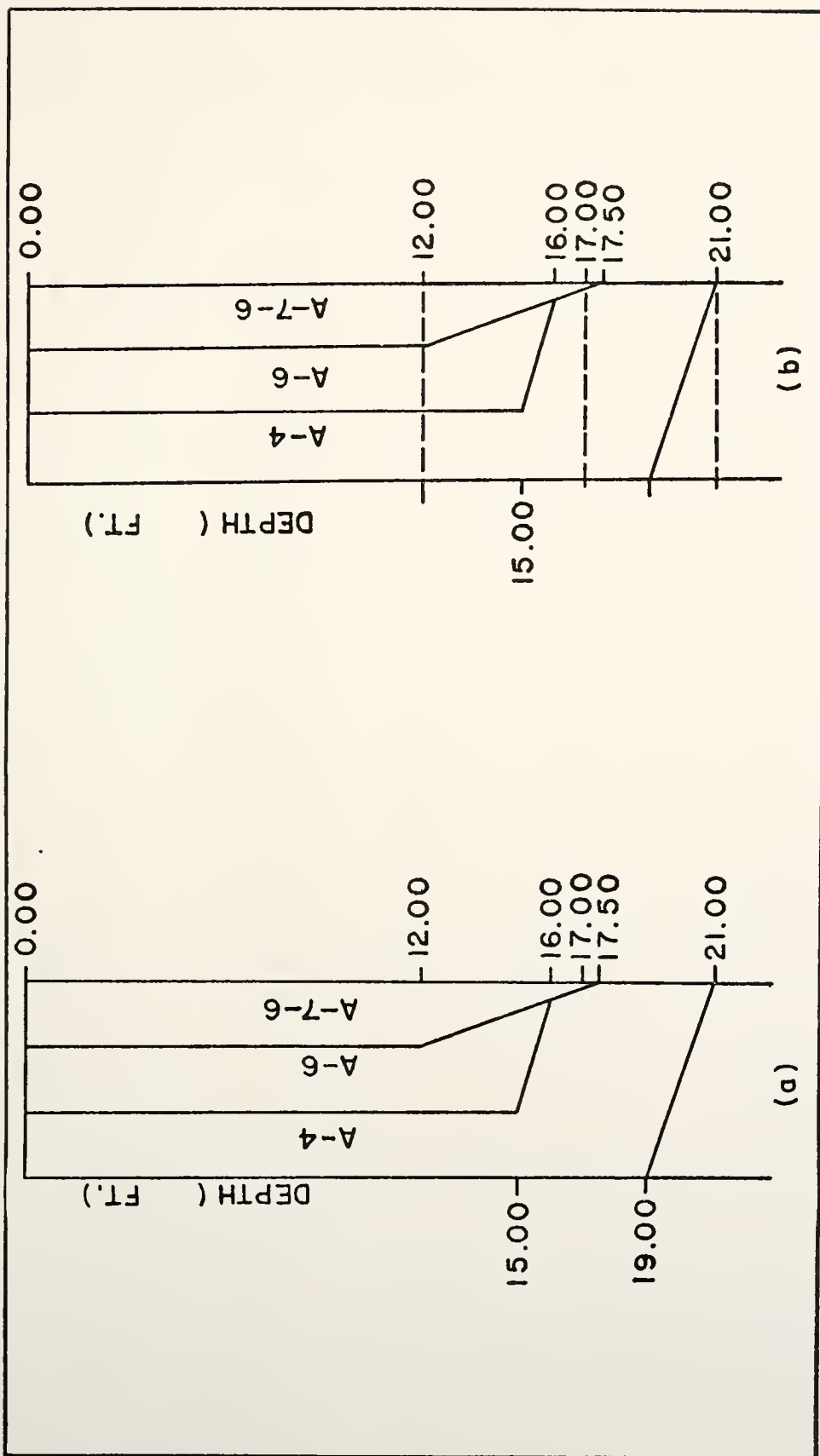


Figure 3-3 Construction of a Soil Layer System





Table 3-17. AASHTO Units vs. DEPTH T for Soil Association Wellston-Zanesville-Berks

AASHTO DEPTH T	A-1-A	A-1-B	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-4	A-5	A-6	A-7-5	A-7-6	A-8
(ft)													
0		1	5					16		25	1	13	
1			1					7		5		7	
2			2					7		6		7	
3		1	2					6		4		7	
4			2		2			10		9	1	8	
5			1					1		2	2	5	
6			1					5		2	1	6	
7										1		1	
8			2							2		4	
9			2							3	2	4	
10								1		1		2	
11								2		4		3	
12					1					3	1		
13										2	2	1	
14								4		1			
15								2		2		2	
16										3			
17			1										
18								5		2		1	
19													

Note: Only the "dominate" (page 52) soils are used...A-4, A-6, A-7-6.







The first layer ranges from 1 foot\* to 12 feet, the second layer from 12 feet to 17 feet, and the third layer from 17 feet to 21 feet, as shown in Figure 3-3(b). All the upper boundaries of A-4, A-6, and A-7-6 soils extend to a depth of 12 feet. Further divisions of layers may be necessary. Referring to Figure 3-3(b) and Table 3-17, and for the sake of simplicity, the upper 12 foot layer is divided into half. Therefore a four layer soil profile system is established. The first layer is from 1 foot to 6 feet, the second layer from 6 feet to 12 feet, the third layer from 12 feet to 17 feet, and the fourth layer from 17 feet to 21 feet.

The soil characteristics presented in each layer are shrinkage limit in % (SL), natural moisture content ( $w_n$ ), natural dry density ( $\rho_d$ ), specific gravity (G), maximum dry density ( $\rho_{d \max}$ ), optimum moisture content (OMC), CBR soaked value at 100% maximum dry density (CBR SO 1), CBR soaked value at 95% maximum dry density (CBR SO2), and unconfined compressive strength ( $q_u$ ). The sample distribution of each soil characteristic for a specific layer was then examined by using the procedures discussed in Section 3.3.1.a. All available data are used, regardless of their AASHTO classification units.

---

\*The top one foot of soil was regarded as top soil.



In addition to the soil characteristics mentioned above, estimates of sample distributions of SPT, preconsolidation pressure ( $p_c$ ), cohesive strength intercept ( $c$ ) and strength angle ( $\phi$ ) were examined and added to the statistical soil profiles (Table 3-18).

Thirty-eight statistical soil profiles were generated using the above procedures. The results are accumulated in Appendix A-111.

### 3.3.2.a. Regression Models

Regression analysis provides a conceptually simple method for investigating functional relationships among variables. In general, the first stage of the analysis is to select the variables to be included in the regression model. This is done based upon theory, on former examples, or by other procedures.

The most thorough approach, known as the all possible regression method, is to develop the regression of  $y$  (dependent variable) on every subset of the  $k$   $x$  variables (independent variables). The major drawback of this method is the amount of computation. Another approach for selecting variables, and the one used in this study, is the stepwise regression method. For details refer to Chatterjee et al (18) and Draper et al (26). It is recommended (18) that the step-wise procedures be applied only to noncollinear data, and the order in which the variables enter or leave the equation not be interpreted





Table 3-18.

# Statistical Soil Profile for the Soil Association Wellston - Zanesville - Berks

Soil Association: Wellston-Zanesville-Berks

General Description: Sloping, well drained, silty Wellston and sloping, well drained, silty Zanesville with fragipans, both in wind-blown silts and weathered sandstone and shale, and steep Berks in weathered sandstone and shale.

Parent Material:

Soils formed in residuum from siltstone, shale and sandstone bedrock.

Distribution:

Physiographic unit: Crawford Upland

Counties: Crawford, Dubois, Perry.

Topographic Characteristics:

Ground elevation ( ft.):

mean: 541.23	s.d.: 110.85	median: 529.09
t. <sub>.25</sub> : 457.50	t. <sub>.75</sub> : 620.00	I.R.: 162.50
95% C.I. of median: 482.50~565.45	No. of cases: 59	
minimum: 422.50	maximum: 803.80	

Ground water elevation ( ft.):

mean: 532.87	s.d.: 107.88	median: 519.54
t. <sub>.25</sub> : 454.47	t. <sub>.75</sub> : 615.00	I.R.: 160.53
95% C.I. of median: 480.00~555.91	No. of cases: 59	
minimum: 417.20	maximum: 784.80	

Water depth with relation to ground elevation ( ft.):

mean: 8.36	s.d.: 7.12	median: 5.86
t. <sub>.25</sub> : 3.22	t. <sub>.75</sub> : 11.67	I.R.: 8.45
95% C.I. of median: 4.26~6.63	No. of cases: 59	
minimum: 1.00	maximum: 37.00	

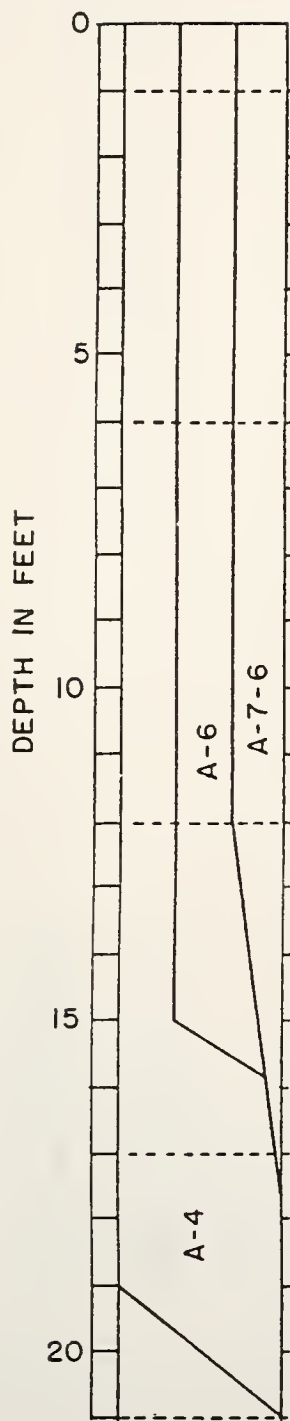




Table 18. (continued)

Engineering Characteristics:

Depth: 0.00' - 1.00'

Unified Classification: CL

PH: 4.5 - 5.0

Texture: silty clay loam (A-4, A-6, A-7-6)

Organic Material: not traceable

Depth: 0.00' - 6.00'

Unified Classification: CL, CH

Texture: clay (A-6, A-7-6), silty clay loam  
(A-4, A-6, A-7-6)

Organic Material: not traceable

PH: 4.5 - 5.0

Variable	Mean	S.D.	Median	t <sub>.25</sub>	t <sub>.75</sub>	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (z)	18.55	3.81	18.13	15.24	21.18	5.94	17.11 -- 19.11	10.50	28.00	142
W <sub>n</sub> (z)	22.17	4.77	21.87	18.33	25.00	6.67	20.55 -- 23.19	11.60	34.70	88
ρ <sub>d</sub> (pcf)	103.73	7.12	103.42	100.13	108.25	8.12	101.58 -- 105.50	88.00	120.40	49
C	2.71	0.04	2.72	2.70	2.73	0.03	2.67 -- 2.74	2.60	2.75	10
ρ <sub>dmax</sub> (pcf)	107.26	4.40	107.80	104.00	110.00	6.00	104.00 -- 110.00	100.80	116.00	14
OMC (z)	18.16	2.48	18.75	17.00	20.75	3.75	17.00 -- 20.75	14.30	22.00	14
CBR S01 (z)	7.13	3.39	7.00	5.00	10.00	5.00	5.00 -- 10.00	1.40	14.60	14
CBR S02 (z)	4.08	1.82	3.83	2.67	5.50	2.83	2.67 -- 5.50	1.30	7.60	14
q <sub>u</sub> (Tsrf)	0.86	0.67	0.67	0.35	1.10	0.75	0.40 -- 1.00	0.20	3.10	33



Table 3-18. (continued)

Depth: 6.00' - 12.00'

Unified Classification: CL, CH

PH: 4.5 - 5.0

Texture: clay (A-6, A-7-6), silty clay loam, sandy loam (A-4, A-6, A-7-6)  
Organic Material:

Variable	Mean	S.D.	Median	t <sub>.25</sub>	t <sub>.75</sub>	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (%)	16.50	4.35	16.32	12.71	19.15	6.44	15.00 -- 18.46	10.00	26.00	56
W <sub>n</sub> (%)	23.36	9.14	21.00	16.67	30.00	13.33	17.78 -- 26.67	12.00	43.30	27
ρ <sub>d</sub> (pcf)	97.50	14.84	98.75	90.00	112.50	22.50	90.00 -- 112.50	71.00	117.60	14
G	2.69	0.06	2.70	2.67	2.74	0.07	--	2.65	2.76	4
ρ <sub>dmax</sub> (pcf)	104.67	4.20	105.00	102.50	107.50	5.00	--	100.20	109.30	4
OMC (%)	20.05	2.84	20.00	18.00	22.50	4.50	--	17.50	23.40	4
CBR SO1 (%)	4.07	1.12	4.55	3.50	4.65	1.15	--	2.40	4.70	4
CBR SO2 (%)	2.00	0.89	2.30	1.50	2.50	1.00	--	0.70	2.70	4
q <sub>u</sub> (Tsf)	1.15	0.86	0.90	0.57	1.60	1.03	0.57 -- 1.60	0.50	3.20	8



Table 3-18. (continued)

Depth: 12.00' - 17.00'

Texture: clay (A-6, A-7-6), silty clay loam (A-4, A-6, A-7-6)

Unified Classification: CL

Organic Material: not traceable

PH: 4.5 - 5.00

Variable	Mean	S.D.	Median	t <sub>.25</sub>	t <sub>.75</sub>	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (z)	18.87	4.90	18.90	15.67	22.36	6.69	16.67 -- 21.54	10.00	27.00	32
W <sub>n</sub> (z)	27.65	5.74	28.50	23.75	32.50	8.75	23.75 -- 32.50	17.80	39.30	18
ρ <sub>d</sub> (pcf)	94.29	7.50	96.25	90.83	101.25	10.42	90.00 -- 101.25	77.20	107.00	13
C	2.70	0.02	2.70	2.69	2.71	0.02	--	2.68	2.72	6
ρ <sub>dmax</sub> (pcf)	107.00	--	107.00	--	--	--	--	--	--	1
OMC (z)	17.50	--	17.50	--	--	--	--	--	--	1
CBR S01 (z)	4.70	--	4.70	--	--	--	--	--	--	1
CBR S02 (z)	2.70	--	2.70	--	--	--	--	--	--	1
q <sub>u</sub> (Tsf)	0.56	0.26	0.55	0.28	0.81	0.53	--	0.20	0.90	5





Table 3-J8. (continued)

Depth: 17.00 - 21.00'

Texture: silty clay loam (A-4)

Unified Classification: CL, CL-ML

Organic Material: not traceable

PH: 4.5 - 5.00

Variable	Mean	S.D.	Median	t <sub>.25</sub>	t <sub>.75</sub>	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (Z)	21.65	2.61	22.00	19.50	23.12	3.62	21.00 -- 23.00	16.00	23.40	17
W <sub>n</sub> (Z)	27.04	3.26	27.73	25.68	29.77	4.09	25.91 -- 29.54	19.30	31.00	17
$\rho_d$ (pcf)	97.96	4.82	97.50	94.00	101.67	7.67	94.00 -- 101.67	91.20	109.10	14
C	2.71	0.02	2.71	2.70	2.72	0.02	--	2.68	2.72	6
$\rho_{dmax}$ (pcf)	--	--	--	--	--	--	--	--	--	0
OMC (Z)	--	--	--	--	--	--	--	--	--	0
CBR SO1 (Z)	--	--	--	--	--	--	--	--	--	0
CBR SO2 (Z)	--	--	--	--	--	--	--	--	--	0
q <sub>u</sub> (Tsf)	0.80	0.28	0.80	--	--	--	--	0.60	1.00	2



Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	P <sub>c</sub>	c <sub>v</sub>	Type	c	φ
13.	-0	2.00	3.	-0	-0	-0	-0	-0
13.	-0	2.00	-0	-0	-0	3.	.25	21.0
13.	-0	2.00	-0	-0	-0	3.	.15	24.0
13.	-0	3.00	7.	-0	-0	-0	-0	-0
13.	8.5	4.25	14.	-0	-0	-0	-0	-0
13.	-0	4.25	13.	-0	-0	-0	-0	-0
13.	-0	4.25	-0	-0	-0	3.	.50	31.0
13.	-0	4.75	28.	-0	-0	-0	-0	-0
13.	-0	5.50	5.	-0	-0	-0	-0	-0
13.	-0	6.00	6.	-0	-0	-0	-0	-0
13.	-0	6.75	5.	-0	-0	-0	-0	-0
13.	-0	7.00	7.	-0	-0	-0	-0	-0
13.	-0	7.00	4.	-0	-0	-0	-0	-0
13.	18.0	7.75	6.	-0	-0	-0	-0	-0
13.	-0	9.75	44.	-0	-0	-0	-0	-0
13.	-0	11.75	80.	-0	-0	-0	-0	-0
13.	-0	12.75	10.	-0	-0	-0	-0	-0
13.	-0	12.75	-0	-0	-0	3.	0	30.0
13.	24.0	13.25	25.	-0	-0	-0	-0	-0
13.	16.5	13.75	61.	-0	-0	-0	-0	-0



Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	P <sub>c</sub>	c <sub>v</sub>	Type	c	φ
13.	16.5	13.75	61.	-0	-0	-0	-0	-0
13.	-0	14.00	-0	-0	-0	3.	.10	20.0
13.	22.5	14.75	-0	-0	-0	-0	-0	-0
13.	16.5	15.25	31.	-0	-0	-0	-0	-0
13.	-0	15.50	7.	-0	-0	-0	-0	-0
13.	-0	16.00	-0	-0	-0	3.	.25	21.0
13.	25.0	16.75	7.	-0	-0	-0	-0	-0
13.	-0	18.25	45.	-0	-0	-0	-0	-0
19.	-0	3.50	-0	1.60	-0	-0	-0	-0
19.	-0	6.00	-0	1.65	-0	-0	-0	-0
19.	-0	6.00	-0	3.20	-0	-0	-0	-0
19.	-0	6.00	-0	-0	-0	-0	-0	-0
19.	-0	6.00	-0	4.40	-0	-0	0	32.6
19.	-0	6.00	-0	-0	-0	-0	-0	-0
19.	-0	6.00	-0	-0	-0	3.	0	32.6
19.	-0	6.00	-0	-0	-0	3.	0	32.6
19.	9.5	7.25	31.	-0	-0	-0	-0	-0
19.	-0	7.50	5.	-0	-0	-0	-0	-0
19.	-0	7.75	3.	-0	-0	-0	-0	-0



Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	P <sub>c</sub>	c <sub>v</sub>	Type	c	φ
62.	-0	.20	-0	3.00	-0	-0	-0	-0
62.	9.0	1.00	3.	-0	-0	-0	-0	-0
62.	5.5	1.75	18.	-0	-0	-0	-0	-0
62.	-0	1.75	64.	-0	-0	-0	-0	-0
62.	-0	3.00	-0	2.15	-0	-0	-0	-0
62.	-0	3.25	-0	2.10	-0	-0	-0	-0
62.	-0	4.00	-0	1.50	-0	-0	-0	-0
62.	5.5	4.25	28.	-0	-0	-0	-0	-0
62.	9.0	4.25	16.	-0	-0	-0	-0	-0
62.	-0	4.75	2.	-0	-0	-0	-0	-0
62.	9.3	5.75	18.	-0	-0	-0	-0	-0
62.	6.5	5.75	21.	-0	-0	-0	-0	-0
62.	-0	5.75	4.	-0	-0	-0	-0	-0
62.	-0	5.75	2.	-0	-0	-0	-0	-0
62.	8.5	6.75	25.	-0	-0	-0	-0	-0
62.	-0	7.50	-0	2.40	-0	-0	-0	-0
62.	-0	7.75	-0	6.30	-0	-0	-0	-0
62.	-0	8.00	5.	-0	-0	-0	-0	-0





Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	P <sub>c</sub>	c <sub>v</sub>	Type	c	φ
19.	-0	10.25	25.	-0	-0	-0	-0	-0
19.	-0	11.00	-0	2.15	3.60	-0	-0	-0
19.	-0	12.75	9.	-0	-0	-0	-0	-0
19.	-0	12.75	7.	-0	-0	-0	-0	-0
19.	-0	16.00	-0	2.00	-0	-0	-0	-0
19.	32.8	17.00	-0	1.35	3.60	-0	-0	-0
19.	-0	18.00	-0	-0	-0	-0	0	29.3
19.	-0	18.00	-0	-0	-0	-0	0	29.3
19.	-0	18.00	-0	-0	-0	-0	0	29.3
19.	-0	19.25	11.	-0	-0	-0	-0	-0
19.	29.5	19.25	-0	-0	-0	2.	0	36.0
19.	29.5	19.25	-0	-0	-0	2.	0	36.0
19.	29.5	19.25	-0	-0	-0	2.	0	36.0
19.	-0	19.50	-0	3.10	-0	-0	-0	-0
19.	29.5	23.80	-0	-0	-0	2.	0	33.0
19.	29.5	23.80	-0	-0	-0	2.	0	33.0
19.	-0	23.80	-0	-0	-0	2.	0	33.0



Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	P <sub>c</sub>	c <sub>v</sub>	Type	c	φ
62.	-0	8.25	24.	-0	-0	-0	-0	-0
62.	-0	10.75	73.	-0	-0	-0	-0	-0
62.	-0	14.25	51.	-0	-0	-0	-0	-0
62.	22.3	14.75	38.	-0	-0	-0	-0	-0
62.	-0	18.25	9.	-0	-0	-0	-0	-0
62.	18.0	19.25	50.	-0	-0	-0	-0	-0

Note: 1. (-0) unknown

2. County and Type as coded.

3. Bedrk in ft.      Depth in ft.  
     P<sub>c</sub> in Tsf.      c<sub>v</sub> in ft<sup>2</sup>/mth.  
     c in Tsf.      φ in degree.



as reflecting the relative importance of the variables.

A question arises while entering the variables to formulate a regression model, as to the form of each variable, i.e., should it enter the model as an original variable  $x$ , or as some transformed variable such as  $x^2$ ,  $\log x$ , or a combination of both. If from an examination of scatter plots of  $y$  against  $x$  the relationship between  $y$  and  $x$  appears to be nonlinear, appropriate transformations of the data are introduced to produce linearity (18). In this study, all variables, their possible transformations, and their combinations were included in the step-wise procedure for selecting variables, so long as they were not collinear.

The variables were selected to minimize the MSE (mean square due to error) of the prediction. As a large value of  $R^2$  or a significant  $t$  statistic does not insure that the data were well fitted (3), a careful residual analysis was also made. The procedure to reduce the number of independent variables was to compare the full model (FM) and reduced model (RM) by using the  $F$ -statistic. For details of this procedure refer to (18).

It was believed that the soil was more homogeneous in a small geologic or pedologic unit. Therefore, the regression models were established on these units. It was often found that for a given dependent variable the regression models generated in this way used different sets of independent variables for various locations. It



seems unwise to conclude that these differences are caused by soil differences alone.

The effects of soil location and genesis, viz., physiographic region and parent material, were investigated by employing the statistical technique of using qualitative variables as regressors (18). In order to do so, the qualitative variables were represented by dummy variables which take on only two values, usually zero and one. These two values designated whether the observation belonged in one of two possible categories. Accordingly, the number of these variables required was one less than the number of categories in a grouping unit. Reference (53) shows that for Indiana the physiographic regions are coded from 1 to 12 and parent materials from 1 to 13. The dummy variables indicators were set up as follows:

$$x_i = \begin{cases} 1 & \text{if the soil sample is taken from the physiographic region coded as } i \\ 0 & \text{otherwise} \end{cases}$$

where  $i = 1, 2, 3, \dots, 11$ ;

and

$$z_j = \begin{cases} 1 & \text{if the soil sample is derived from the parent material coded as } j \\ 0 & \text{otherwise} \end{cases}$$

where  $j = 1, 2, 3, \dots, 12$ .

For the soil sample taken from the physiographic region coded as 13, let  $x_1 = x_2 = x_3 = \dots = x_{12} = 0$ . And for the soil sample derived from the parent material coded as





12, let  $z_1 = z_2 = z_3 = \dots = z_{11} = 0$ .

Assume that the following relationship exists:

$$C_c = a_0 + a_1 w_n + a_2 e_o + a_3 w_L$$

where  $C_c$  is the compression index,  $w_n$  the natural moisture content (%),  $e_o$  the initial void ratio, and  $w_L$  the liquid limit (%). To investigate how these two soil grouping units affect this relationship singly or in combination, the F-statistics were employed for making comparisons of the following models:

$$\text{Model 1: } C_c = a_0 + a_1 w_n + a_2 e_o + a_3 w_L$$

$$\begin{aligned} \text{Model 2: } C_c = & a'_0 + a'_1 w_n + a'_2 e_o + a'_3 w_L + a'_4 x_1 + a'_5 x_2 \\ & + \dots + a'_{15} x_{12} \end{aligned}$$

$$\begin{aligned} \text{Model 3: } C_c = & a''_0 + a''_1 w_n + a''_2 e_o + a''_3 w_L + a''_4 z_1 + a''_5 z_2 \\ & + \dots + a''_{14} z_{11} \end{aligned}$$

and

$$\begin{aligned} \text{Model 4: } C_c = & a'''_0 + a'''_1 w_n + a'''_2 e_o + a'''_3 w_L + a'''_4 x_1 \\ & + a'''_5 x_2 + \dots + a'''_{15} x_{12} + a'''_{16} z_1 + a'''_{17} z_2 \\ & + \dots + a'''_{26} z_{11}. \end{aligned}$$

### 3.3.2.b. The Application of Regression Models

The regression models were used to correlate soil design parameters, such as those of compaction, consolidation, and strength, with soil index properties. The following example illustrates how to apply the procedures described in the previous section to arrive at the regression model for maximum dry density ( $\rho_{d \max}$ ) using index properties.



Table 3-19 shows the correlation coefficients (R) between the  $\rho_{d \text{ max}}$  and selected index properties. It shows that the  $\rho_{d \text{ max}}$  is well correlated with  $w_L$  and  $w_p$ . Scatter plots are shown in Figures 3-4 and 3-5. In the presentation of the data, the solid line represents the best fit line while the dashed lines define the boundaries of 95% population confidence interval. Examinations of these two scatter plots indicate that seemingly curvilinear relationships exist between  $\rho_{d \text{ max}}$  and  $w_L$  and  $w_p$ . There should also be an examination of whether the interaction between  $w_L$  and  $w_p$ , i.e.,  $w_L \cdot w_p$ , has any significant

Table 3-19 Correlation Table for Maximum Dry Density ( $\rho_{d \text{ max}}$ ) Versus Some Index Properties

Coefficient of Correlation (R)	Natural Moisture Content, %, ( $w_n$ )	Natural Dry Density, pcf, ( $\rho_d$ )	% Sand by Weight (sand)
$\rho_{d \text{ max}}$ (pcf)	- 0.525 (n = 241)	0.714 (n = 20)	0.201 (n = 642)

Table 3-19 (continued)

	% Silt by Weight, (silt)	Plastic Limit, % ( $w_p$ )	Liquid Limit %, ( $w_L$ )
	-0.243 (n = 668)	-0.665 (n = 623)	-0.720 (n = 625)



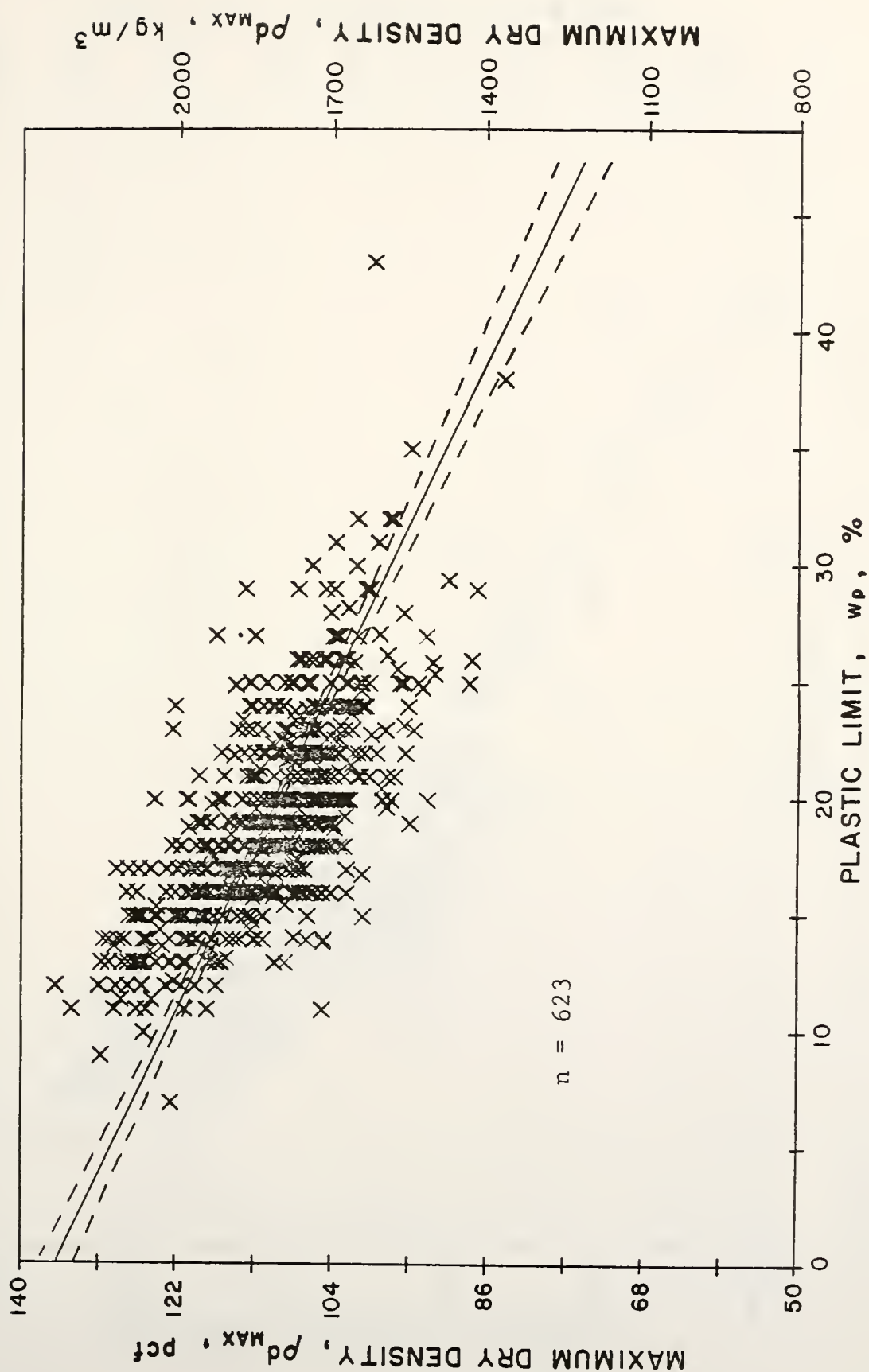


Figure 3-4. Maximum Dry Density vs Plastic Limit, Indiana Soils



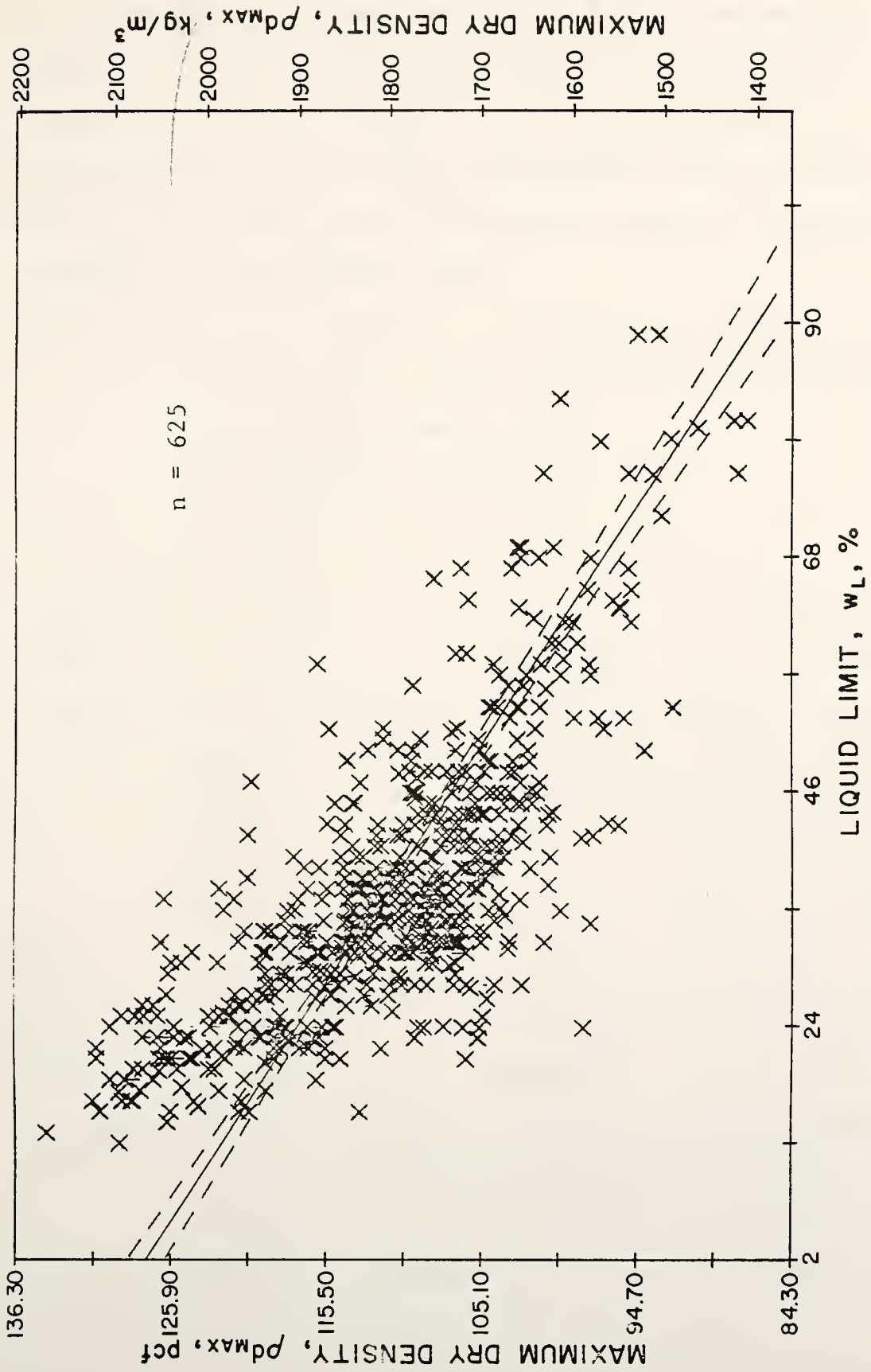


Figure 3-5. Maximum Dry Density vs. Liquid Limit, Indiana Soils.





effect on  $\rho_{d \max}$ . Therefore, the following independent variables enter the step-wise regression program for consideration:  $w_L$ ,  $w_L^2$ ,  $w_p$ ,  $w_p^2$ ,  $w_L \cdot w_p$ , % sand and % silt. Since there were not enough corresponding data available for  $w_n$  and  $\rho_d$ , they do not enter the program as regressor candidates. The results follow.

Model 1:  $\hat{\rho}_{d \max} = -0.463 w_L + 128.337$ , for which  $|R| = 0.743$ , standard deviation of estimate = 5.651, and  $n = 601$ .

ANOVA (analysis of variance) table

source	d.f.	s.s.	m.s.
regression	1	23619.122	23619.122
errors	599	19131.922	31.940

Model 2:  $\hat{\rho}_{d \max} = -0.465 w_L - 0.138 \text{ silt} + 133.995$ , for which  $|R| = 0.791$ , standard deviation of estimate = 5.176, and  $n = 601$ .

ANOVA table

source	d.f.	s.s.	m.s.
regression	2	26726.969	13363.485
errors	598	16024.075	26.796

Model 3:  $\hat{\rho}_{d \max} = -0.371 w_L - 0.105 \text{ silt} - 0.396 w_p + 136.794$ , for which  $|R| = 0.803$ , standard deviation of estimate = 5.047, and  $n = 601$ .



ANOVA table

source	d.f.	s.s.	m.s.
regression	3	27544.092	9181.364
errors	397	15206.952	25.472

Model 4:  $\hat{\rho}_{d \max} = -0.554 w_L - 0.0900 \text{ silt} - 0.277 w_p$   
 $+ 0.00849 w_L \cdot w_p + 142.888$ , for which  $|R|$   
 $= 0.808$ , standard deviation of estimate  
 $= 4.984$ , and  $n = 601$ .

ANOVA table

source	d.f.	s.s.	m.s.
regression	4	27884.774	6971.193
errors	596	14866.270	24.943

Model 5:  $\hat{\rho}_{d \max} = -0.587 w_L - 0.0898 \text{ silt} - 0.608 w_p$   
 $+ 0.0100 w_L w_p - 0.00430 w_p^2 + 142.344$ , for which  
 $|R| = 0.808$ , standard deviation of estimate  
 $= 4.998$ , and  $n = 601$ .

ANOVA table

source	d.f.	s.s.	m.s.
regression	5	27890.335	5578.067
errors	595	14860.709	24.976

Model 6:  $\hat{\rho}_{d \max} = -0.591 w_L - 0.0897 \text{ silt} - 0.606 w_p$   
 $+ 0.0122 w_L w_p - 0.00648 w_p^2 - 0.000470 w_L^2 + 142.404$ ,



for which  $|R| = 0.808$ , standard deviation of estimate = 5.001, and  $n = 601$ .

ANOVA table

source	d.f.	s.s.	m.s.
regression	6	27892.792	4648.799
errors	594	14858.252	25.014

Model 7:  $\hat{p}_{d \max} = -0.588 w_L - 0.0858 \text{ silt} - 0.608 w_p$   
 $+ 0.0126 w_L w_p - 0.00683 w_p^2 - 0.000546 w_L^2$   
 $+ 0.00585 \text{ sand} + 142.000$ , for which  $|R|$   
 $= 0.808$ , standard deviation of estimate  
 $= 5.005$ , and  $n = 601$ .

ANOVA table

source	d.f.	s.s.	m.s.
regression	7	27894.718	3984.960
errors	593	14856.326	25.053

The F-statistic tests were employed to reduce the number of independent variables as follows:

Case i, Model 7 vs 4

$H_0$ : coefficients of sand,  $w_L^2$ ,  $w_p^2 = 0$ ,  $H_A$ : not  $H_0$ .

$$F_{3,583} = \frac{(27894.718 - 27884.774)/(7-4)}{25.053}$$

$= 0.132 < 8.54$  at 5% level.

$H_0$  is accepted, i.e., the variables of sand,  $w_L^2$ , and  $w_p^2$  do not add significant information to Model 4.



case ii, Model 4 vs 3:

$H_0$ : coefficient of  $w_L w_p = 0$ ,  $H_A$ : not  $H_0$

$$F_{1,596} = \frac{(27884.774 - 27544.092)/(4-3)}{24.943}$$

$$= 13.658 > 3.86 \text{ at } 5\% \text{ level.}$$

$H_0$  is rejected, i.e., the variable  $w_L w_p$  does add significant information to Model 3.

case iii, Model 3 vs 2:

$H_0$ : coefficient of  $w_p = 0$ ,  $H_A$ : not  $H_0$

$$F_{1,597} = \frac{(27544.092 - 26726.969)/(3-2)}{25.472}$$

$$= 32.079 > 3.86 \text{ at } 5\% \text{ level.}$$

$H_0$  is rejected.

Case iv, Model 2 vs 1:

$H_0$ : coefficient of silt = 0,  $H_A$ : not  $H_0$

$$F_{1,598} = \frac{(26726.969 - 23619.122)/(2-1)}{26.796}$$

$$= 115.982 > 3.86 \text{ at } 5\% \text{ level}$$

$H_0$  is rejected.

Hence, the final model was in the form of:

$$\hat{\rho}_{d \max} = -0.554 w_L - 0.0900 \text{ silt} - 0.727 w_p + 0.00849 w_L w_p + 142.888, \text{ Equation 3.3,}$$

for which  $|R| = 0.808$ , standard deviation of estimate

$= 4.994$ , and  $n = 601$ . For a given predicted  $\rho_{d \max}$

$(\hat{\rho}_{d \max})$  about 68% of sample observations (measured

$\rho_{d \max}$ ) fall in the range of  $\hat{\rho}_{d \max} + 4.994$  and  $\hat{\rho}_{d \max}$

$- 4.994$ .





To investigate the effects of physiographic regions and parent materials on Equation 3.3 the dummy variable indicators were set up as follows:

$$x_i = \begin{cases} 1 & \text{if the soil sample is taken from the physiographic region coded as } i \\ 0 & \text{otherwise} \end{cases}$$

where  $i = 1, 2, 3, \dots, 11^*$

and

$$z_j = \begin{cases} 1 & \text{if the soil sample is derived from the parent material coded as } j \\ 0 & \text{otherwise} \end{cases}$$

where  $j = 1, 2, 3, \dots, 11$ .

Adding these dummy variable indicators to the regression model (Equation 3.3) for further analysis, the regression models were developed as follows.

Model A:  $\hat{\rho}_{d \max} = -0.554 w_L - 0.0900 \text{ silt} - 0.727 w_p + 0.00849 w_L w_p + 142.888$ , for which  $|R| = 0.808$ , standard deviation of estimate = 4,994, and  $n = 601$ .

ANOV table

source	d.f.	s.s	m.s.
regression	4	27884.774	6971.193
errors	596	14866.270	24.943

---

\*There were no soil samples available from the Maumee Lacustrine Section, coded as 12.



Model B:  $\hat{\rho}_{d \max} = -0.546 w_L - 0.0903 \text{ silt} - 0.680 w_p$   
 $+ 0.00818 w_L w_p + 0.540 x_1 + 0.292 x_2 + 4.450 x_3$   
 $- 1.992 x_4 + 0.488 x_5 - 1.374 x_6 + 0.368 x_7 - 0.138 x_8$   
 $- 1.574 x_9 - 0.136 x_{10} + 0.755 x_{11} + 141.852$   
 for which  $|R| = 0.814$ , standard deviation of  
 estimate = 4.960, and  $n = 601$ .

## ANOVA table

source	d.f.	s.s.	m.s.
regression	15	28359.223	1890.615
errors	585	14391.821	24.601

Model C:  $\hat{\rho}_{d \max} = -0.564 w_L - 0.0920 \text{ silt} - 0.695 w_p$   
 $+ 0.00873 w_L w_p - 2.625 z_1 - 3.912 z_2 - 6.532 z_3$   
 $- 4.281 z_4 - 2.706 z_5 - 4.147 z_6 - 2.717 z_7$   
 $- 3.0270 z_8 - 4.314 z_9 - 3.390 z_{10} - 4.449 z_{11}$   
 $+ 146.137$

for which  $|R| = 0.813$ , standard deviation of  
 estimate = 4.976, and  $n = 601$ .

## ANOVA table

source	d.f.	s.s.	m.s.
regression	15	28263.429	1884.228
errors	585	14487.615	24.765

Model D:  $\hat{\rho}_{d \max} = -0.563 w_L - 0.0938 \text{ silt} - 0.654 w_p$   
 $+ 0.00828 w_L w_p + 0.240 x_1 + 0.191 x_2 + 4.516 x_3$   
 $- 2.677 x_4 - 0.478 x_5 - 2.453 x_6 - 0.664 x_7 - 0.447 x_8$



$$\begin{aligned}
& - 2.501 x_9 + 0.569 x_{10} + 0.281 x_{11} - 2.480 z_1 \\
& - 4.761 z_2 - 7.246 z_3 - 3.746 z_4 - 3.715 z_5 \\
& - 5.321 z_6 - 3.847 z_7 - 4.190 z_8 - 4.684 z_9 \\
& - 3.329 z_{10} - 3.254 z_{11}
\end{aligned}$$

for which  $|R| = 0.819$ , standard deviation of estimate = 4.946, and  $n = 601$ .

ANOV table

source	d.f.	s.s.	m.s.
regression	26	28710.798	1104.261
errors	574	14040.246	24.460

The F statistic tests were again used to examine the effects of physiographic regions and parent materials on the Model A, i.e., Equation 3.3

Case i: Model D vs A

$H_0$ : coefficient of x's and z's = 0,  $H_A$ : not  $H_0$ .

$$F_{22,574} = \frac{(28710.798 - 27884.774)/(26-4)}{24.460}$$

$$= 1.535 < 1.560 \text{ at } 5\% \text{ level.}$$

$H_0$  is accepted, i.e., neither physiographic regions nor parent materials add any significant information on Model A.

Case ii: Model B vs A

$H_0$ : coefficient of x's = 0,  $H_A$ : not  $H_0$ .

$$F_{11,585} = \frac{(28359.223 - 27884.774)/(15-4)}{24.601}$$

$$= 1.753 < 1.810 \text{ at } 5\% \text{ level.}$$



$H_0$  is accepted, i.e., physiographic regions do not add significant information on Model A.

Case iii: Model C vs A

$H_0$ : coefficient of  $z$ 's = 0,  $H_A$ : not  $H_0$ :

$$F_{11,585} = \frac{(28263.429 - 27884.774)/(15-4)}{24.765}$$

$$= 1.390 < 1.810 \text{ at } 5\% \text{ level}$$

$H_0$  is accepted, i.e., parent materials do not add significant information on Model A.

Therefore, equation 3.3 was the final model for maximum dry density ( $\rho_{d \text{ max}}$ ). For the soil variable of compression index ( $C_c$ ) it was found that effects of both physiographic regions and parent materials do add significant information statistically to the regression model of  $C_c$  on index properties. The regression model was found to be

$$\begin{aligned} \hat{C}_c = & -0.151 + 0.00326 w_n + 0.191 e_o + 0.00325 w_L \\ & + 0.0162 x_1 - 0.0110 x_2 + 0.0208 x_3 + 0.0296 x_4 \\ & + 0.0120 x_5 - 0.0110 x_6 + 0.0365 x_7 + 0.0351 x_8 \\ & + 0.0646 x_9 + 0.0649 x_{10} - 0.0594 x_{11} - 0.0245 z_1 \\ & - 0.0313 z_2 - 0.00987 z_3 - 0.0917 z_4 - 0.121 z_6 \\ & - 0.0292 z_7 - 0.0667 z_8 + 0.00841 z_9 - 0.0418 z_{10} \\ & - 0.00884 z_{11} \end{aligned}$$

Equation 3.4\*

for which  $|R| = 0.952$ , standard deviation of estimate = 0.0670, and  $n = 302$ .

---

\*Note that there were no soil sample taken from the Maumee Lacustrine section, coded as 12 in the group of physiographic regions, nor from loamy Wisconsin age glacial till, coded as 5 in the group of parent materials.





The Crawford Upland soils are coded as 7, i.e.,  $x_7$ , in the group of physiographic regions, and those of soil association Wellston-Zanesville-Berks which is the residuum from siltstone, shale and sandstone are coded as 10, i.e.,  $z_{10}$ , in the group of parent materials. With  $x_7 = 1$ ,  $z_{10} = 1$ , and the rest of dummy variables indicators as zero, Equation 3.4 becomes

$$\begin{aligned}\hat{C}_c = & - 0.151 + 0.00326 w_n + 0.191 e_o + 0.00325 w_L \\ & + 0.0365 - 0.0418\end{aligned}$$

$$\text{or} \quad = - 0.156 + 0.00326 w_n + 0.191 e_o + 0.00325 w_L.$$

Figures 3-6 and 3-7 show the scatter plots with regression lines and 95% population confidence intervals for the measured  $\rho_{d \max}$  ( $\rho_{d \max}$ ) and predicted  $\rho_{d \max}$  ( $\hat{\rho}_{d \max}$ ) using Equation 3.3, and the measured  $C_c$  ( $C_c$ ) and predicted  $C_c$  ( $\hat{C}_c$ ) using Equation 3.4. In the presentation of the data, the solid line represents the best fit line, while the dashed lines define the boundaries of 95% population confidence intervals. All the regression models generated in this work are collected in Appendix A-IV.



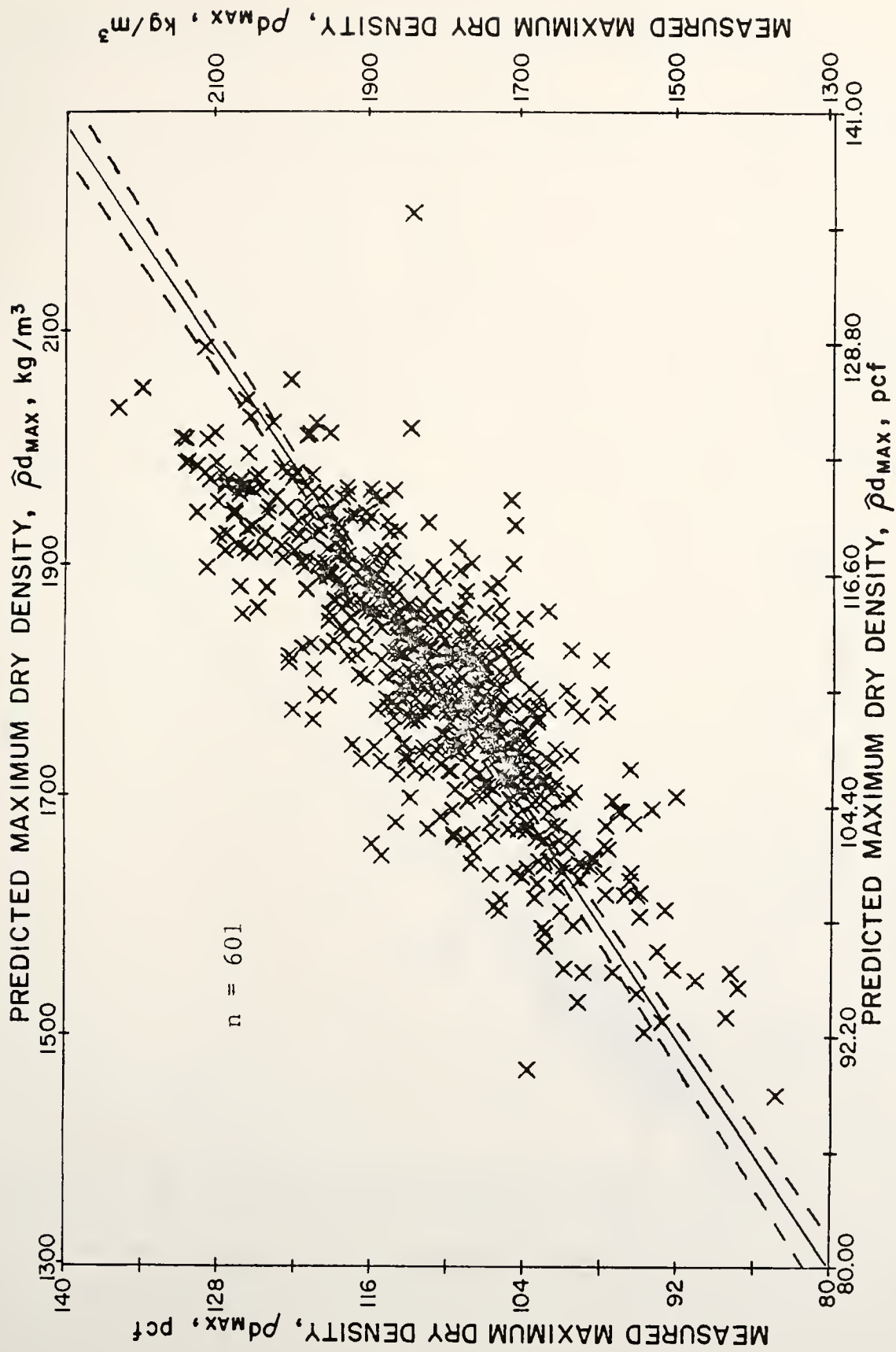


Figure 3-5. Measured Maximum Dry Density vs. Predicted Maximum Dry Density, Indiana Soils



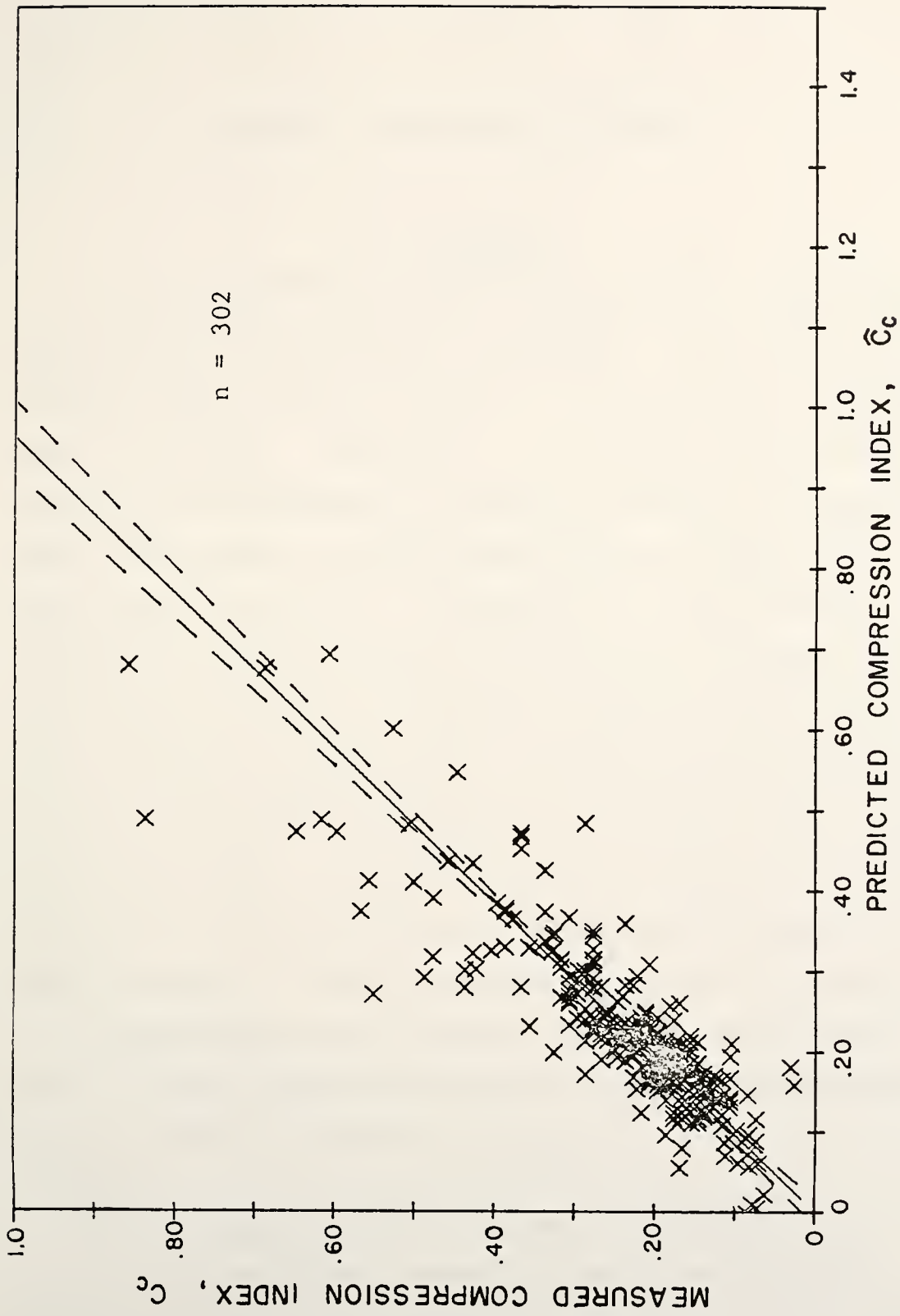


Figure 3-7. Measured Compression Index vs Predicted Compression Index,  
Indiana Soils



## CHAPTER 4 DISCUSSIONS OF RESULTS

### 4.1. General

In this study both median and regression models were developed for statistical forecasting. Predictions are extrapolations into the future of features shown by relevant data in the past. Therefore, a considerable population of values for the dependent variable is required. Another basic requirement for prediction is the existence of a stable data structure. The trend of the data and the statistical variation about the trend must be stable, which can be detected by the confidence intervals and interquartile range for a median model, or the standard deviation of estimate and multiple correlation coefficient ( $|R|$ ) for a regression model. A large difference between the confidence intervals, or a large value of interquartile range for a median model, or a large standard deviation of estimate for a regression model, indicates that the data structure is not stable. Either more data or a change in grouping unit is needed for better prediction.

### 4.2. The Median Model and Soil Variability

In this study soils were grouped by physiographic regions, AASHTO classifications, soil associations, or a





combination of these. The sample distributions of soil characteristics were studied according to these groups. The median of the sample distribution of a soil characteristic was used as the design value.

Technically, the grouping layout used in this study is known as either a one-way classification or a two-way classification. The mathematical model for a median in a one-way classification (Table 4-1) is expressed as

$$\hat{\theta}_{ij} = \theta + \tau_j + e_{ij}, \quad i = 1, \dots, n_j, \quad j = 1, \dots, k,$$

Table 4-1 One-way Classification of Soil Variable ( $\hat{\theta}_{ij}$ )

Soil Groups			
1	2	.	k
$\hat{\theta}_{11}$	$\hat{\theta}_{12}$		$\hat{\theta}_{1k}$
$\hat{\theta}_{21}$	$\hat{\theta}_{22}$		$\hat{\theta}_{2k}$
.	.		.
.	.		.
.	.		.
$\hat{\theta}_{n_1 1}$	$\hat{\theta}_{n_2 2}$		$\hat{\theta}_{n_k k}$

Data: The data consist of  $N = \sum_{j=1}^k n_j$  samples with  $n_j$  samples from the  $j$ -th soil group,  $j = 1, \dots, k$ .

where  $\hat{\theta}_{ij}$  is the overall median,  $\tau_j$  the deviation due to the effect of soil group  $j$ , and  $e_{ij}$  the random error. If  $\tau_1 = \tau_2 = \dots = \tau_k = 0$ , the deviations between the medians of the  $k$ -group soil samples are due only to the random



errors. However, if  $\tau_j \neq \tau_{j+1}$ , then  $\hat{\theta}_{ij} \neq \hat{\theta}_{i j + 1}$ , viz., the deviation between the medians of the  $j$ -th and  $(j+1)$ -th soil sample groups is due to the groupings. The test hypotheses can be set up as follows:

$H_0$ : The medians of the  $k$  populations are equal.

$H_A$ : At least one of the populations has a median different from the others.

In the following, the nonparametric sample comparison methods, such as the median test (101), the Kolmogorov-Smirnov two sample test (42, 101), the Mann-Whitney U-Wilcoxon rank sum  $W$  test (42, 101) the Wald-Wolfowitz run test (42, 101), the extended median test (101), and the Kruskal-Wallis one-way analysis of variance (ANOVA) by ranks test (42, 101), are employed to investigate the group effects on the soil characteristics. For the details of these tests refer to Bradley (14), Hollander et al (42), and Siegel (101).

#### 4.2.a. Topographic Characteristics

##### Versus Physiographic Regions

Topographic characteristics were grouped according to physiographic regions. The test results, as shown in Table C-1 to C-14, Appendix C, verify that the topographic features, such as the ground elevations and ground-water elevations, vary with physiographic regions. These test results, together with the numerical information of topographic characteristics as shown in Table A-I-1 and A-I-3,



Appendix A, can also be used to compare the overall topographic features between two physiographic regions. For example, Table C-2 implies that the median of ground elevations in Tipton Till Plain is greater than that of Dearborn Upland. This is further confirmed by the numerical results as shown in Table A-I-1, Appendix A, in which the medians of the ground elevations of the Tipton Till Plain and Dearborn Upland are 776.64 ft. and 737.67 ft. respectively. Table A-I-1 also shows that the interquartile range (IR) of the Tipton Till Plain is 129.03 ft. and that of the Dearborn Upland is 199.26 ft. This indicates that the general topography of the Dearborn Upland is more rugged than that of the Tipton Till Plain.

Table C-6 and Table A-I-1 also indicate that the general topography of the Crawford Upland is relatively more elevated and rugged than that of the Wabash Lowland.

It is emphasized that the observations of topographic characteristics are not uniformly distributed throughout a physiographic region. The soil data distribution map (Figure 3-1) should be consulted before attempting any such interpretations.

#### 4.2.b. Remolded Soil Characteristics Versus Physiographic Regions and AASHTO Classifications

As mentioned in Section 3.3.1.b. the sample distributions of remolded soil characteristics were studied according to AASHTO classifications for each physiographic



region. Technically, this is a two-way classification layout, i.e., the grouping effects are due to a combination of AASHTO classifications and physiographic regions. These may be investigated by first examining the effects of physiographic regions, followed by an examination of the effects of AASHTO classifications. The test results, as shown in Tables C-15 to C-24, indicate that the remolded soil characteristics vary with either grouping unit, as exemplified by maximum dry density ( $\rho_{d \text{ max}}$ ) and optimum moisture content (OMC). Therefore, it is reasonable to use these grouping units.

The results in Tables C-16\* and C-21\* imply that: the median of  $\rho_{d \text{ max}}$  of A-4 soils > the median of  $\rho_{d \text{ max}}$  of A-6 soils > the median of  $\rho_{d \text{ max}}$  of A-7-6 soils; and the median of OMC of A-7-6 soils > the median of OMC of A-6 soils > the median of OMC of A-4 soils. The more plastic soils have lower  $\rho_{d \text{ max}}$  and higher OMC values. These facts are further verified by use of the two-sample comparison tests, as shown in Tables C-17 to C-19 and Tables C-22 to C-24, and the numerical results in Tables A-II-1 to A-II-36.

#### 4.2.c. Soil Characteristics Versus

##### Soil Associations

The layout for the generation of a statistical soil profile involves a breakdown of the data for each soil characteristic by physiographic regions, and then by soil

---

\* The reader will need to use the code on page 460 to interpret these tables.





associations. Finally they are grouped into proper layer systems. The test results, as illustrated in Tables C-25 and C-26, show that there are differences of soil characteristics for the groups of soil associations, as exemplified by natural dry density ( $\rho_d$ ) and unconfined compressive strength ( $q_u$ ). However, due to the wide scatter of sample frequencies among cells, i.e., soil associations units, further verification is required.

Tables C-29 and C-30 indicate that the natural dry density ( $\rho_d$ ) does not vary significantly either within the group of soil associations coded as 3, 4, 7 and 9 or within the group of soil associations coded as 105, 106, and 107. This may mean either that: (1) the grouping unit is too refined, or (2) the grouping unit is not refined enough. In the latter case a subgrouping unit, soil series, should be perhaps consulted. This subject needs further research.

#### 4.2.d. Soil Series as a Grouping Unit

Tables C-31 and C-32 show the multiple comparison tests of soil characteristics versus soil series, as exemplified by  $\rho_d$  and  $q_u$ . The wide scatter and the shortage of data can be seen from these two tables. In spite of the small sample sizes of soil characteristics, the comparison tests were made. Tables C-33 through C-35 indicate that the soil characteristics do not vary significantly with soil series. Therefore, the soil series as a grouping unit is indicated to be no better than the soil association. Further study would be required to establish this as a firm conclusion.



### 4.3. Regression Models and Correlations

Regression models were used to correlate the soil design parameters, such as those of compaction, consolidation, and strength, with index properties. In Appendix A each regression equation is represented, together with its correlation coefficient ( $R$ ) or multiple correlation coefficient ( $|R|$ ), standard deviation (or error) of estimate (s.d. of est.), and the number of cases ( $n$ ). The (s.d. of est.) is important, as it represents the variation of estimate ( $\hat{y}$ ), viz. 68% of sample observations ( $y$ ) fall in the range of  $\hat{y} - (\text{s.d. of est.})$  and  $\hat{y} + (\text{s.d. of est.})$ . Specific results are presented below.

#### 4.3.a. Compaction Parameters

The correlations of maximum dry density ( $\rho_{d \text{ max}}$ ) and optimum moisture content (OMC) versus plasticity characteristics, as shown in Appendix A, indicate that as the liquid limit or plastic limit increases, the OMC increases but  $\rho_{d \text{ max}}$  decreases. Also, as the OMC increases, the  $\rho_{d \text{ max}}$  decreases. The explanation of these correlations has been previously discussed in Section 2.2.2.a .

The CBR value is regarded as an indirect measure of strength of compacted soil. The strength of a standard compacted soil is a function of its (maximum) dry density and (optimum) moisture content as proposed by Jorgenson (47) and Weitzel (120). Accordingly, the CBR value is a



function of plastic characteristics which is evidenced by the regression equation for CBR at 100% maximum dry density as shown in Appendix A-IV. A relationship between CBR values at 100% and 95% maximum dry densities is also developed and presented in Appendix A.

#### 4.3.b. Consolidation Parameters

It is found (Appendix A-IV) that the compression index ( $C_c$ ) increases with either liquid limit ( $w_L$ ), or natural moisture content ( $w_n$ ), or initial void ratio ( $e_o$ ). The mathematical relationship between compression ratio ( $C'_r$ ) and compression index ( $C_c$ ) is  $C'_r = \frac{C_c}{1+e_o}$  or  $C_c = (1 + e_o) C'_r$ . The quantity of  $C'_r$  is a linear function of  $e_o$  as shown in Appendix A. Therefore, the  $C_c$  is a function of  $(C'_r)^2$  which is verified by the relationship  $C_c = 0.0844 + 9.121 (C'_r)^2$ , as shown in Appendix A-IV.

The recompression index ( $C_r$ ) is a rebound parameter of a clay soil. In practice it is usually taken as a fraction of  $C_c$ . In this study it is found that  $C_r = A + B C_c$  where  $A$  is  $-0.00327$ . The standard deviation of  $A$  is  $0.00199$ ,  $B$  is  $0.139$ , and the standard deviation of  $B$  is  $0.00726$ . Therefore, with 95% confidence the  $C_r$  will lie in the range of, approximately,  $\frac{1}{6.5} C_c$  and  $\frac{1}{8} C_c$  for Indiana soils.

As mentioned in Section 2.2.1.c the preconsolidation pressure ( $p_c$ ) was correlated with liquidity index (LI). Figure 4.1 shows the scatter in this relationship. In the



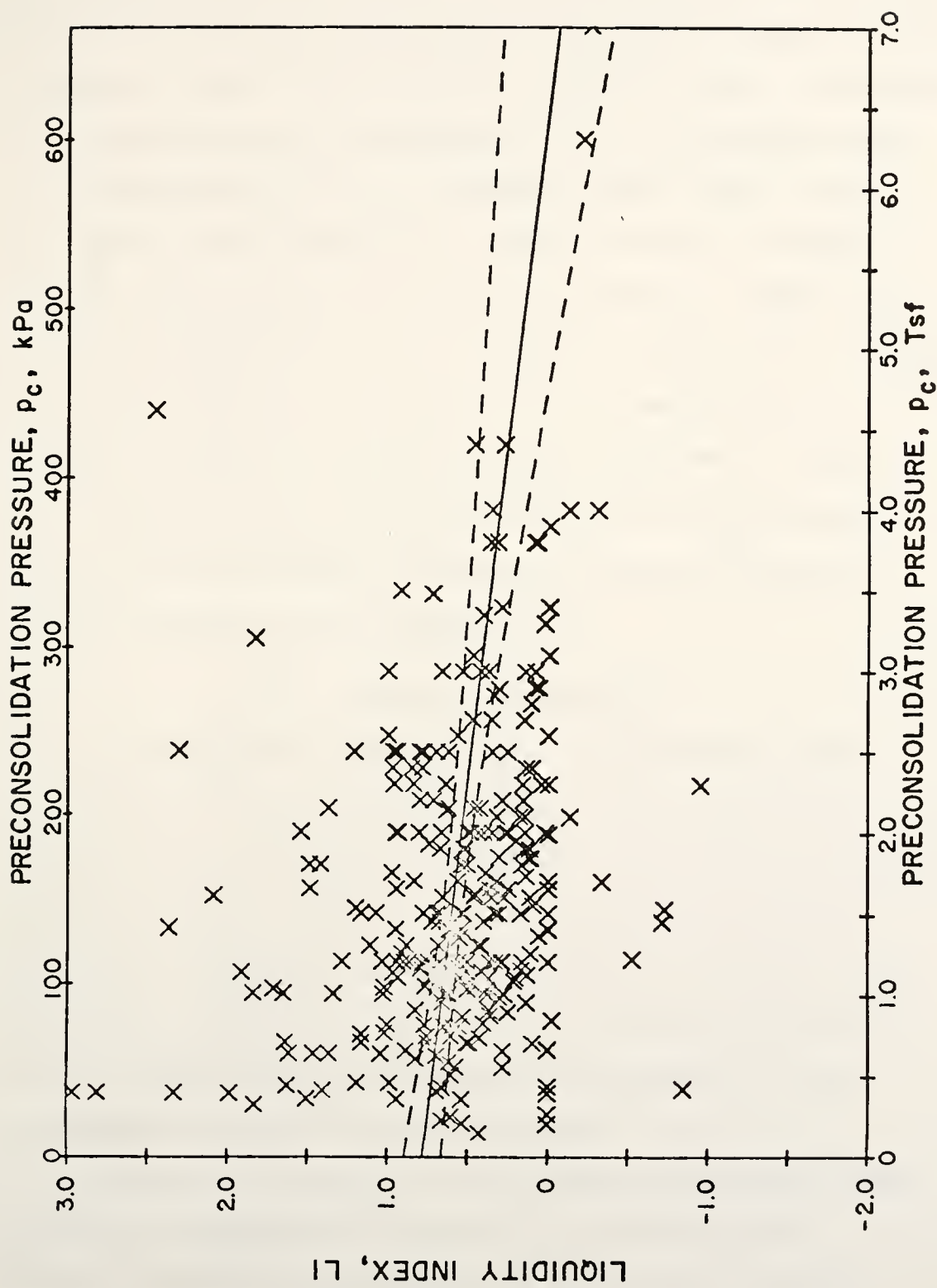


Figure 4-1 Preconsolidation Pressure vs Liquidity Index, Indiana Soils





presentation of the data, the solid line represents the best fit line while the dashed lines define the boundaries of 95% population confidence interval. Therefore, this correlation is not a strong one for Indiana soils. A regression equation was developed for  $p_c$  (Appendix A-IV), which shows the functional relationship of  $p_c$  versus moisture content, initial void ratio, and natural dry density. Again, the scatter in the data is quite large, as indicated by a large standard deviation of estimate and the scatter plot of measured  $p_c$  versus predicted  $p_c$  ( $\hat{p}_c$ ) as shown in Figure 4-2. A part of the difficulty may lie in determining accurate preconsolidation pressures from consolidation tests conducted in the standard way.

#### 4.3.c. Strength Parameters

As mentioned in Section 2.2.3.c there were many qualifications to the relationships between effective strength angle and cohesion intercept versus the plasticity characteristics. Figures 4-3 to 4-6 show no definite correlation of strength angle or cohesion intercept with plasticity characteristics for either unconsolidated undrained, consolidated undrained (unsaturated), or consolidated undrained (saturated) tests.

Figures 4-7 through 4-9 show that as the log of the unconfined compressive strength ( $q_u$ ) increases, the natural dry density increases but the natural moisture content and the liquidity index decrease. In the



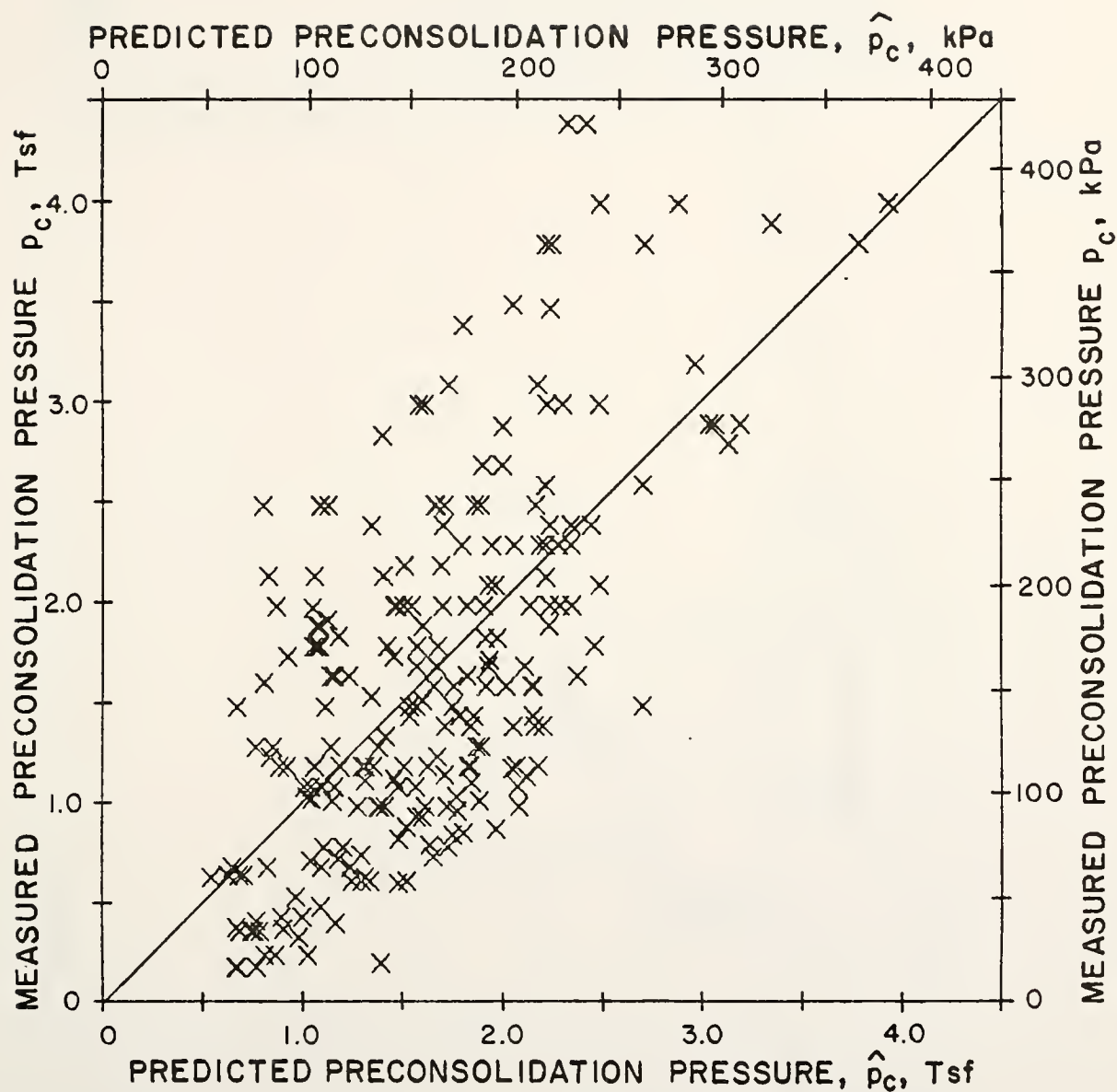
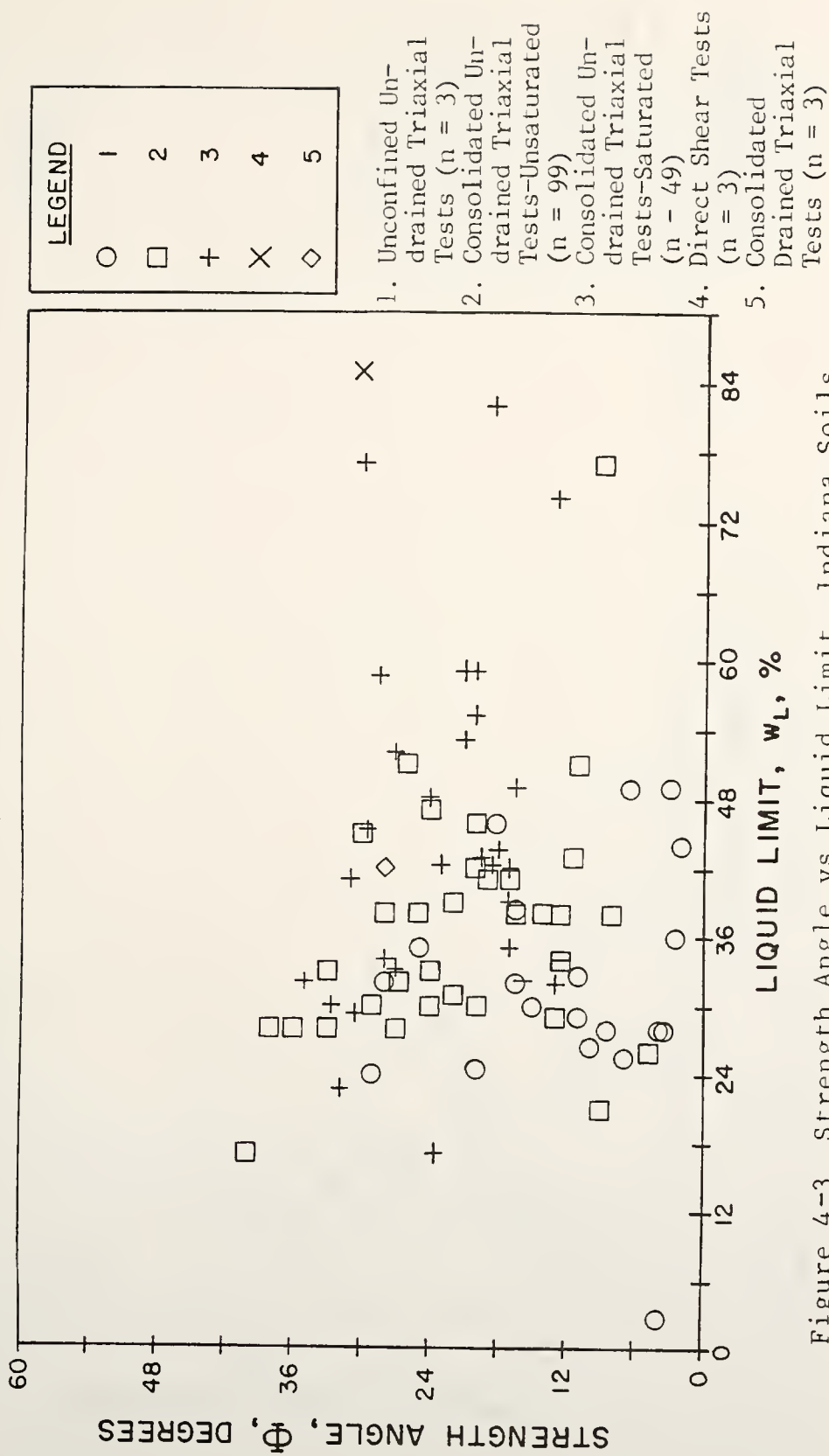


Figure 4-2 Comparison of Measured and Predicted Values of Preconsolidated Pressure (Data from Indiana Geotechnical Bank).







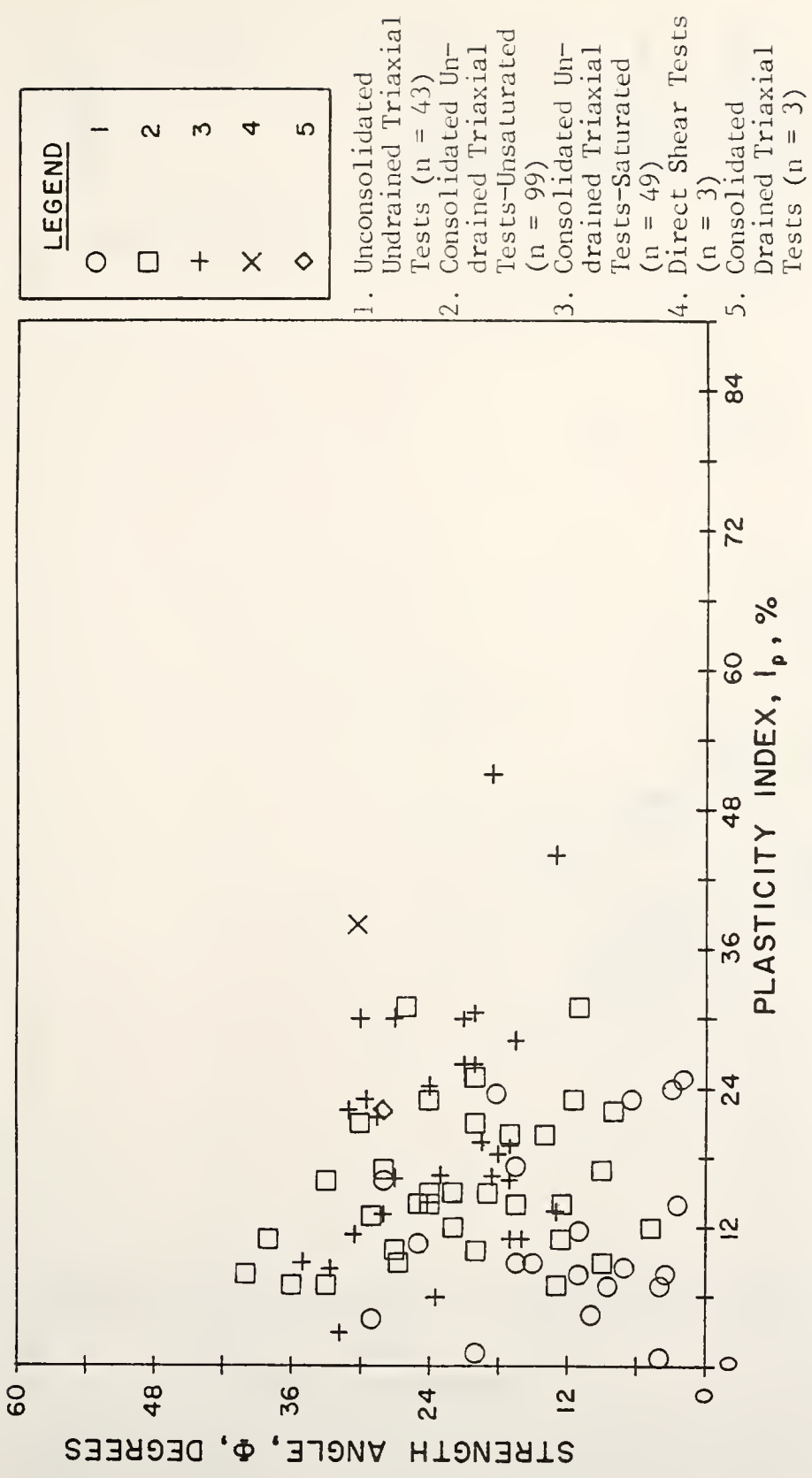


Figure 4-4 Strength Angle vs Plasticity Index, Indiana Soils





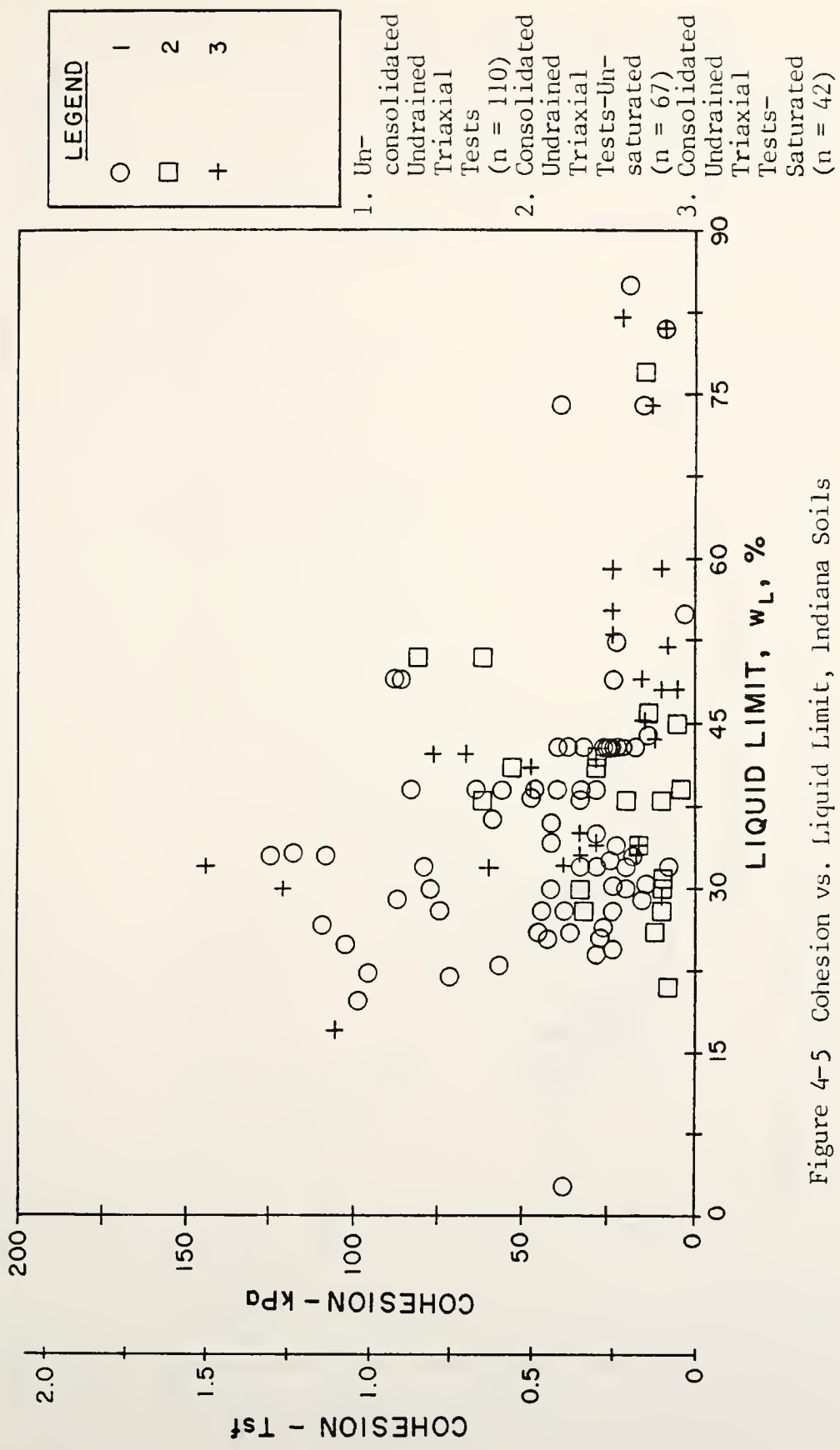


Figure 4-5 Cohesion vs. Liquid Limit, Indiana Soils



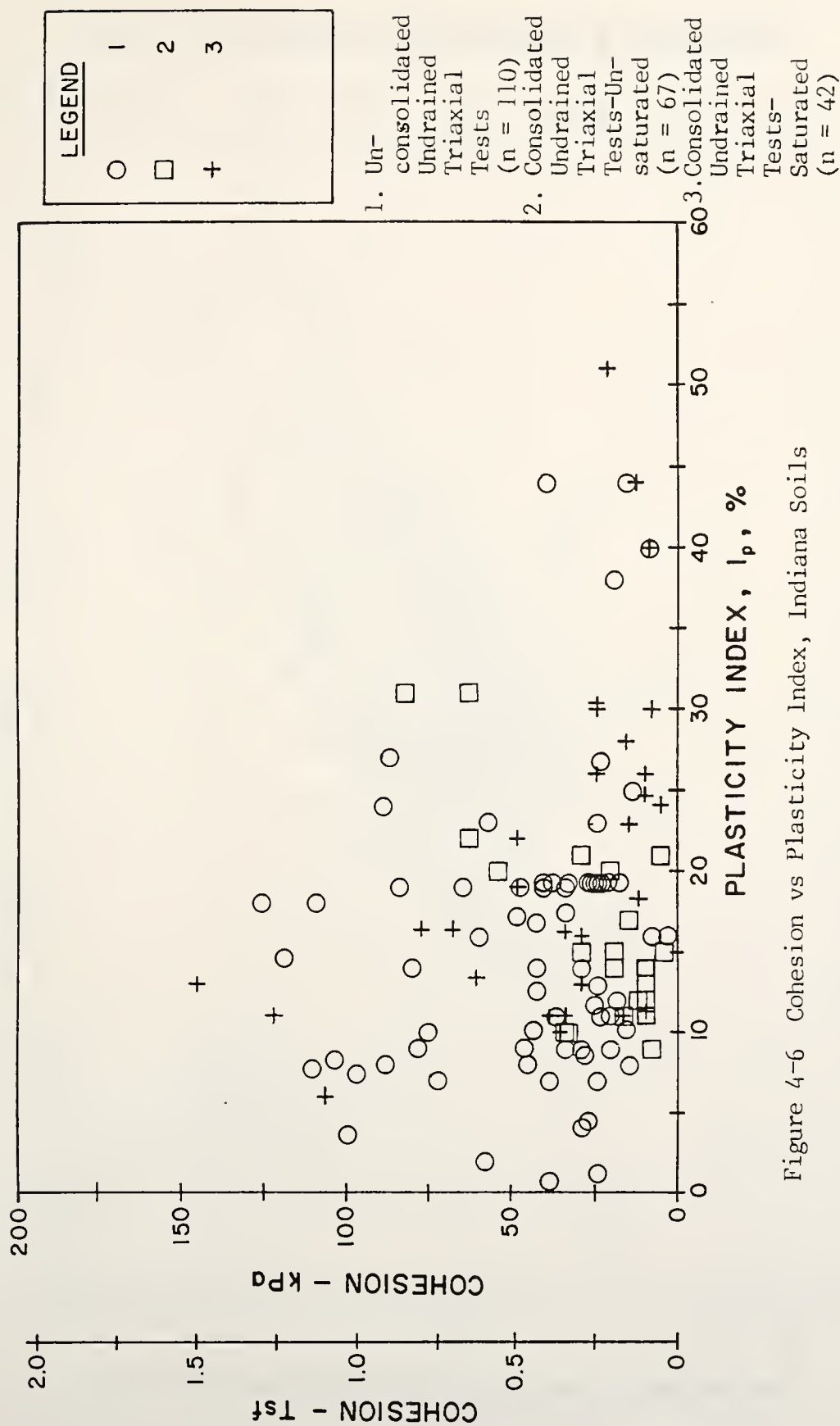


Figure 4-6 Cohesion vs Plasticity Index, Indiana Soils



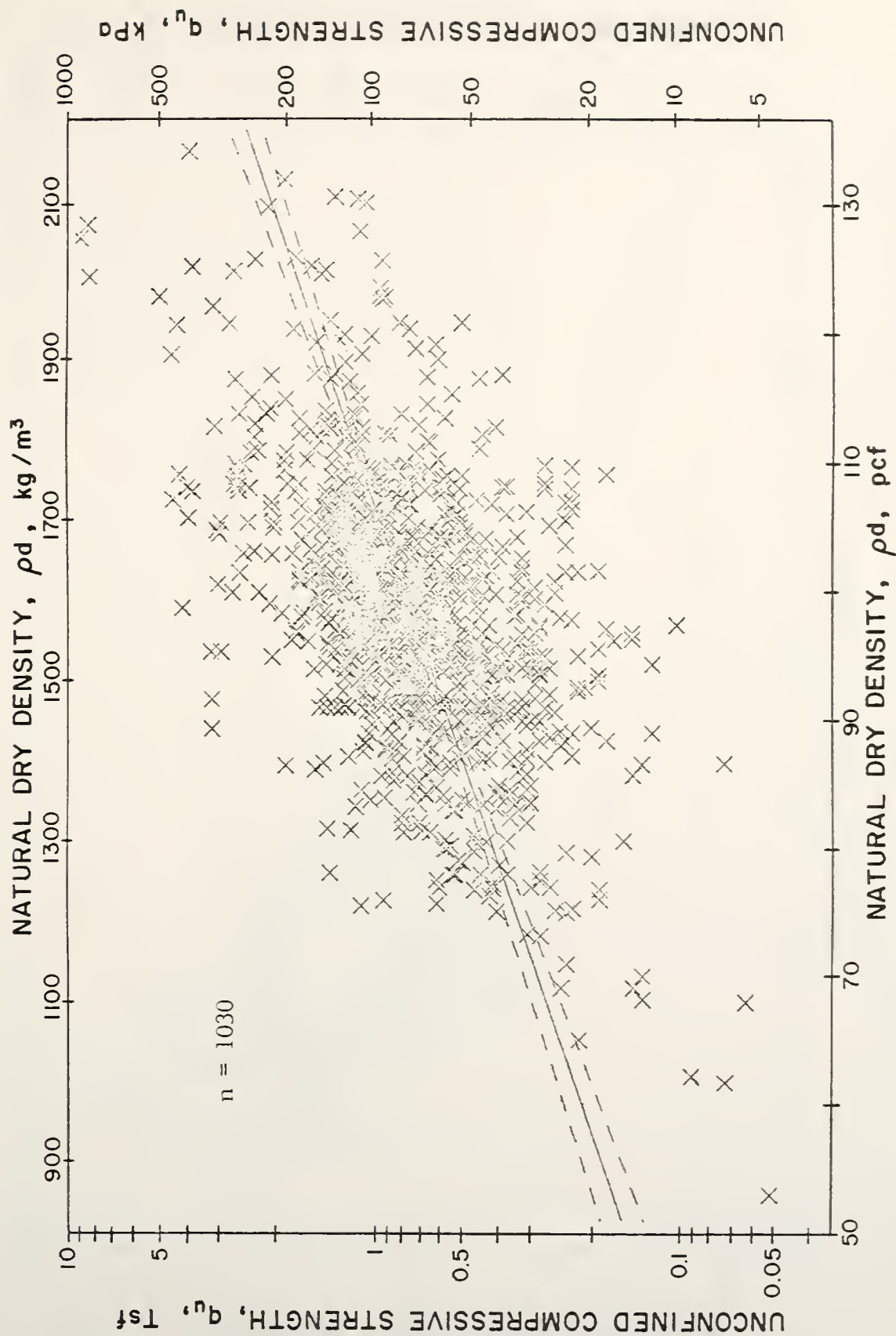


Figure 4-7 Unconfined Compressive Strength vs Natural Dry Density, Indiana Soils



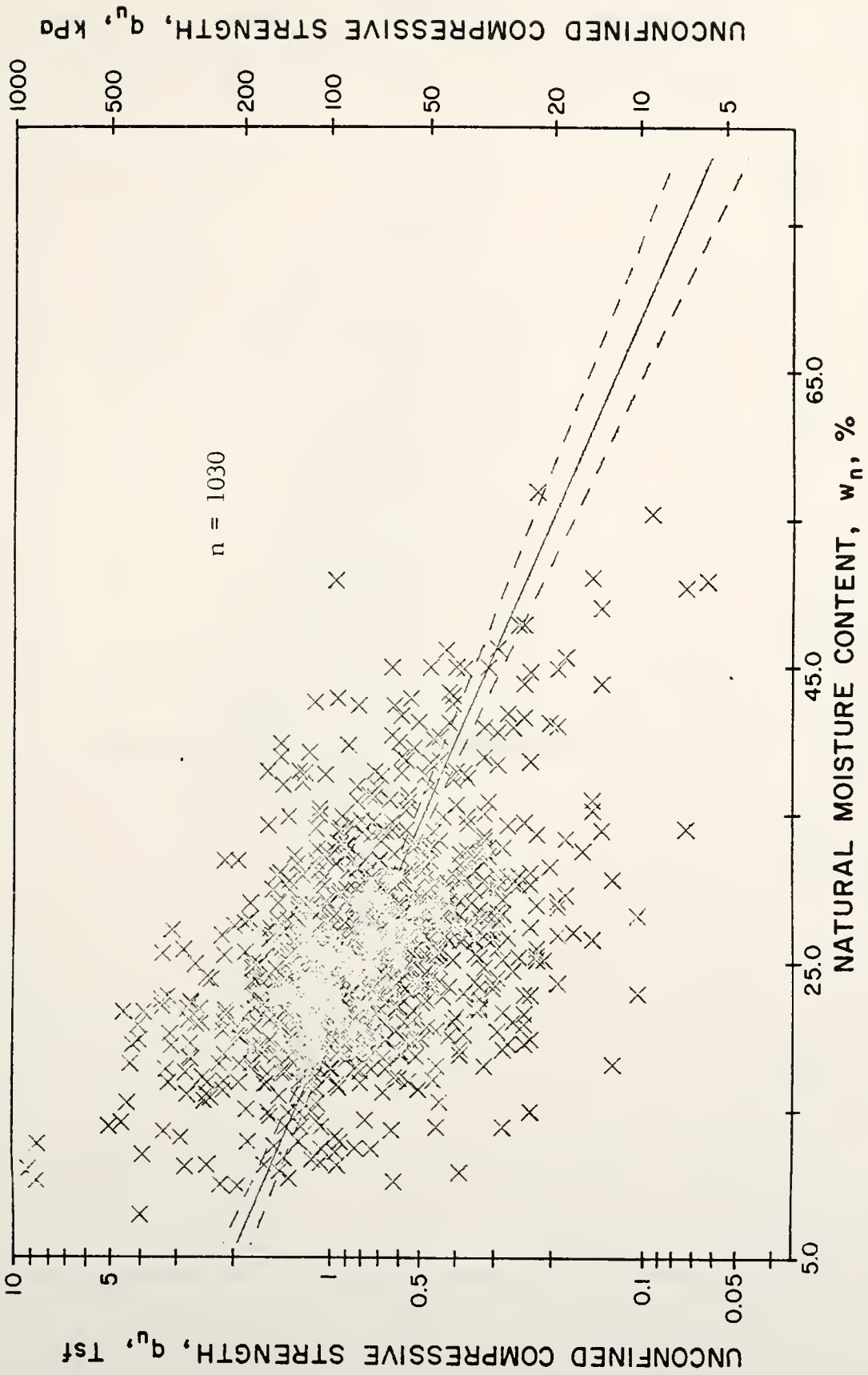


Figure 4-8 Unconfined Compressive Strength vs Natural Moisture Content,  
Indiana Soils





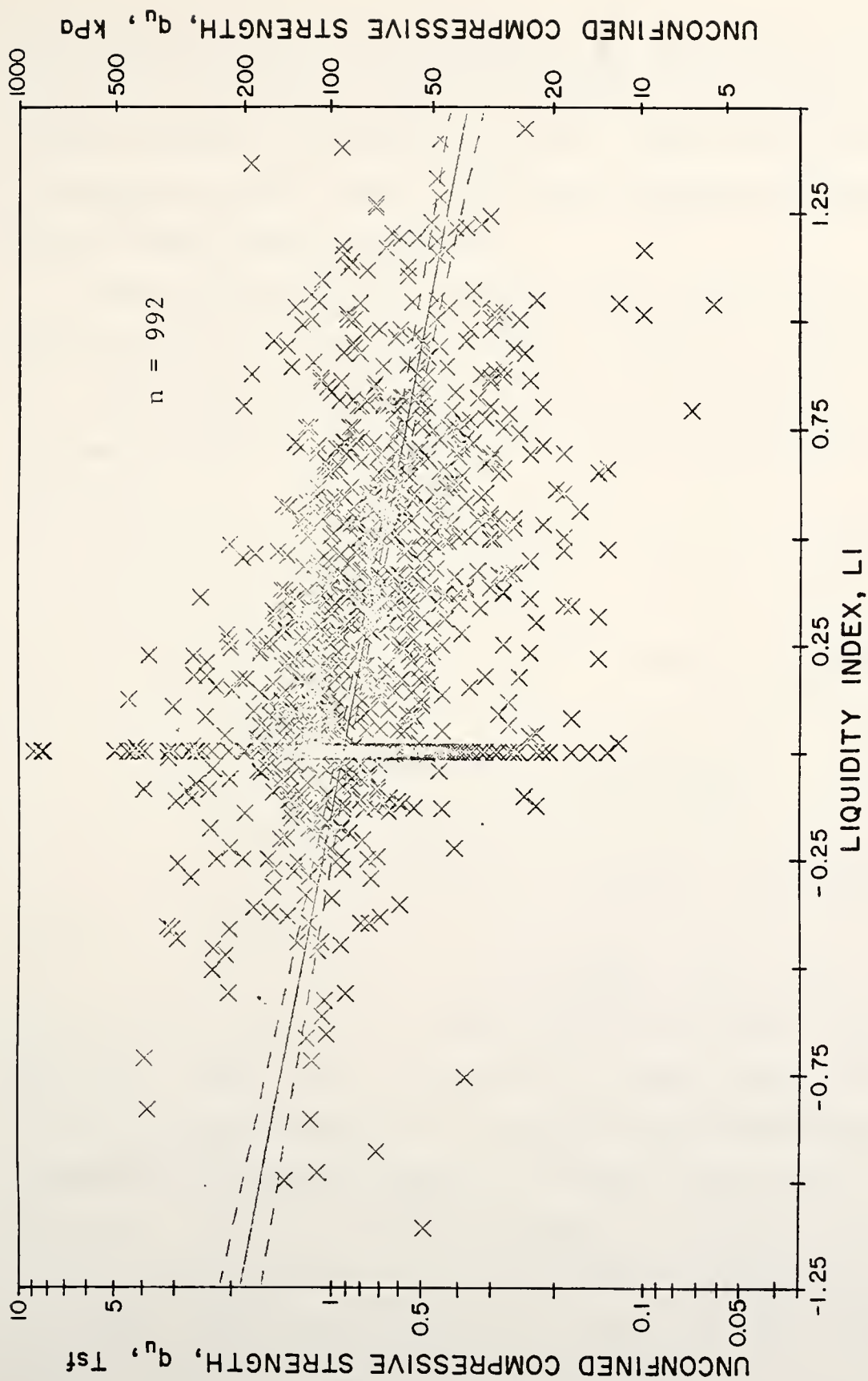


Figure 4-9 Unconfined Compressive Strength vs Liquidity Index, Indiana Soils



presentation of the data, the solid line represents the best fit line while the dashed lines define the boundary or 95% population confidence interval. Since the scatter in the data is quite large, these relationships are qualitative rather than quantitative.

The limit limit ( $w_L$ ) is found to be a function of natural moisture content ( $w_n$ ), SPT, and location factors, i.e., physiographic regions (x's) and parent material (z's) as shown in Appendix A. The equation may be approximately written as

$$\begin{aligned}
 w_L &= 48 + 0.50 w_n - 10 \log_{10} \text{SPT} + .050 w_n \log_{10} \text{SPT} \\
 &\quad + x's + z's \\
 &= w_n - 0.50 w_n - 10 \log_{10} \text{SPT} + 0.50 w_n \log_{10} \text{SPT} + 48 \\
 &\quad + x's + z's \\
 &= w_n - 0.50 w_n \log_{10} 10 + 0.50 w_n \log_{10} \text{SPT} \\
 &\quad - \log_{10} \text{SPT} + \log_{10} 10 + 38 + x's + z's \\
 &= w_n + 0.50 w_n (\log_{10} \text{SPT} - \log_{10} 10) - 10 (\log_{10} \text{SPT} \\
 &\quad - \log_{10} 10) + 38 + x's + z's \\
 &= w_n + (0.50 w_n - 10) \left( \log_{10} \frac{\text{SPT}}{10} \right) + 38 + x's + z's.
 \end{aligned}$$

It can be seen that as the natural moisture content ( $w_n$ ) = 20(%), the liquid limit of the soil is simply equal to 58 + x's + z's. For example, for the association Miami-Russell-Fincastle in Tippecanoe County,  $x_1$  (Tipton Till Plain) = 1,  $x_2 = x_3 = \dots = x_{11} = 0$ ,  $z_8 = 1$ , and  $z_1 = \dots = z_7 = z_9 = \dots = z_{11} = 0$ ,  $w_L = 58 - 19.323 + 23.069 = 61.746(\%)$  at  $w_n = 20(\%)$ , 68(%) of the samples lie within  $61.746 - 19.503 = 42.243 (\%)$  and  $61.746 + 19.503 = 81.249(\%)$ .



Figures 4-10 and 4-11 show the plots of unconfined compressive strength ( $q_u$ ) versus SPT and the strength angle ( $\phi$ ) versus SPT, respectively. In the presentation of the data, the solid line represents the best fit line while the dashed lines define the boundaries of 95% population confidence interval. Since the scatter is large, no reliable predictions can be made.



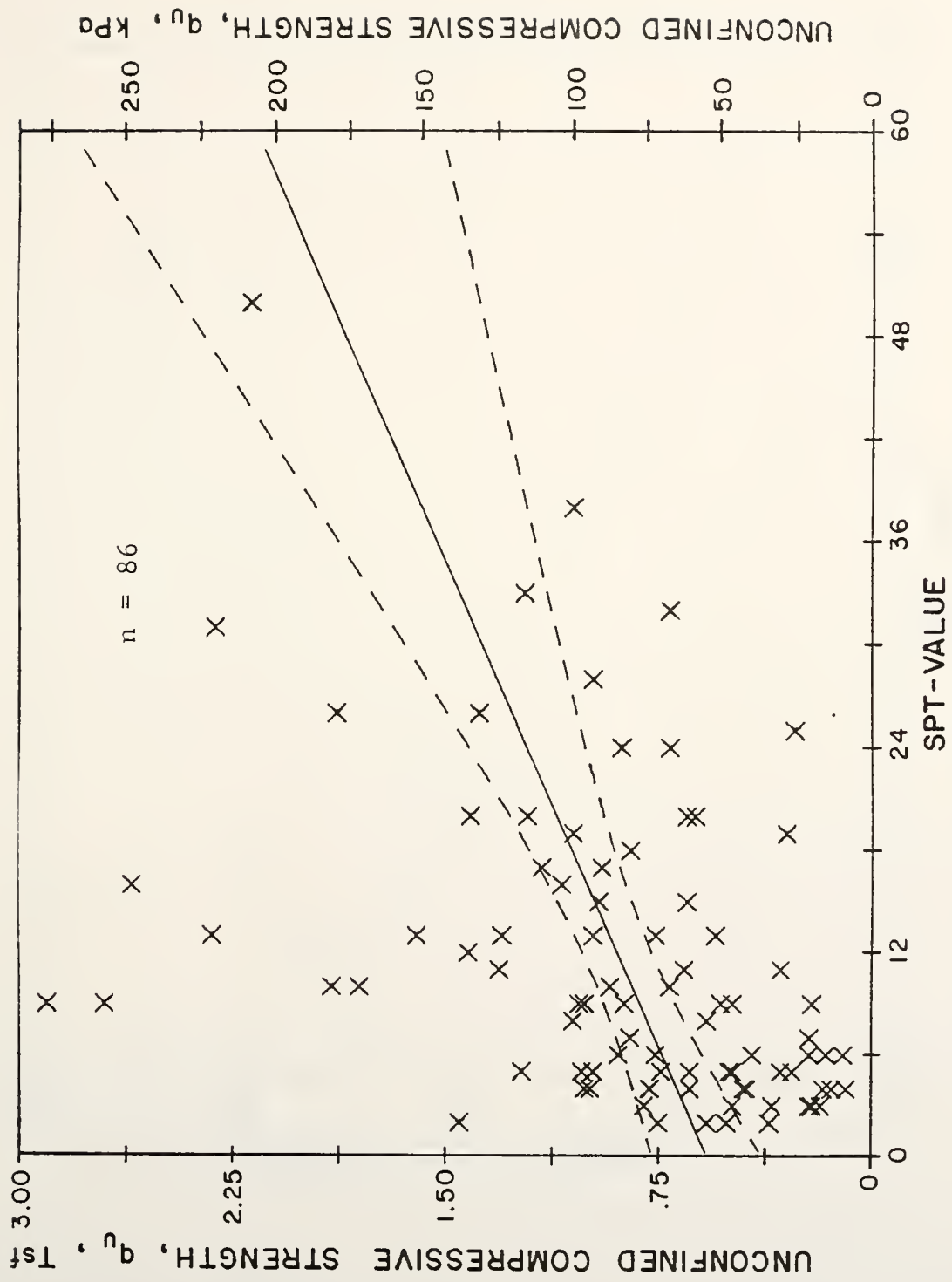


Figure 4-10 Unconfined Compressive Strength vs SPT,  
Indiana Soils





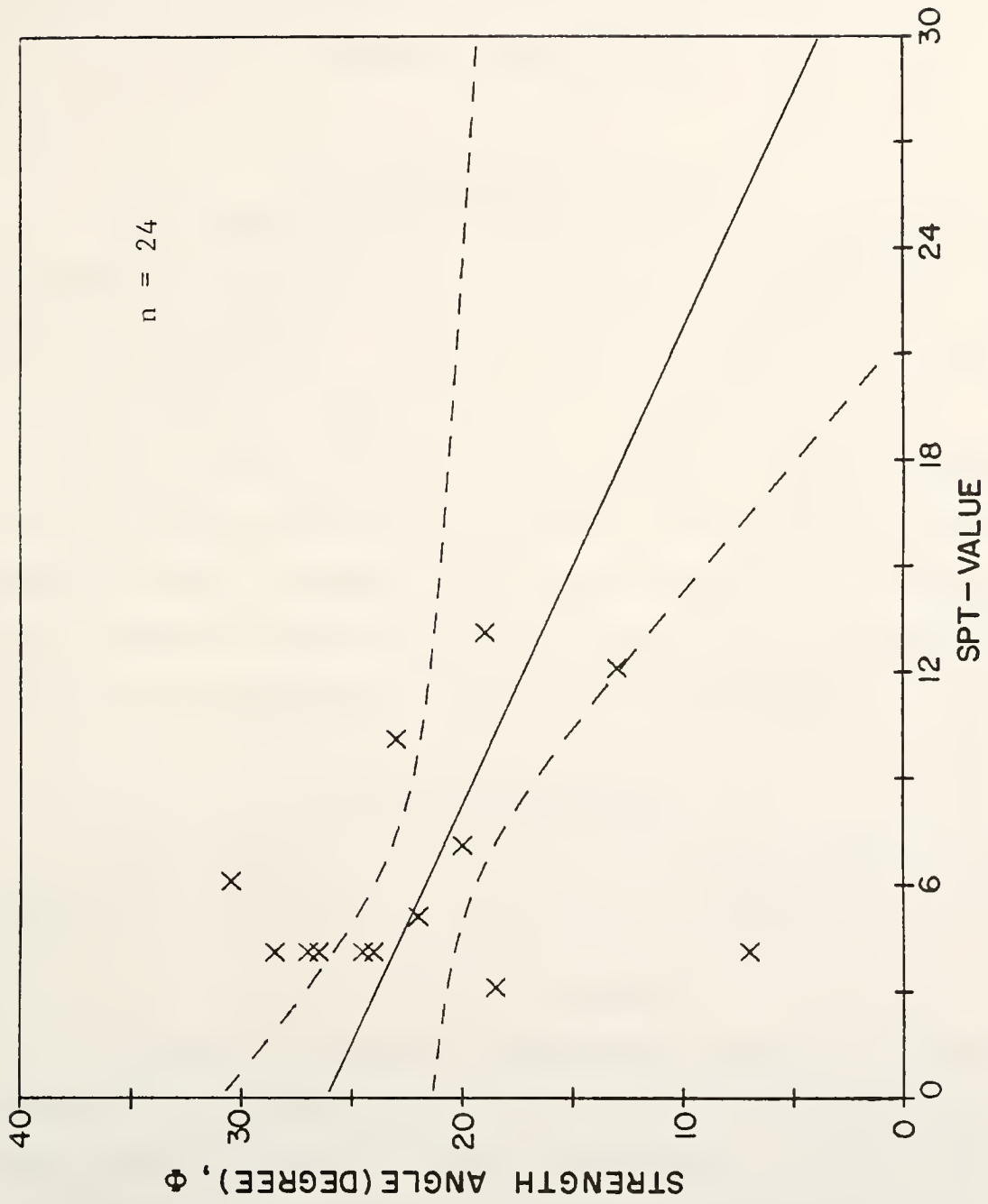


Figure 4-11 Strength Angle vs SPT, Indiana Soils



## CHAPTER 5 APPLICATIONS

### 5.1 Topographic Features

It is important to define the general topographic features of the area of interest before attempting any preliminary design. The topographic features include the ground elevation, water depth with relation to ground elevation, slope, and the depth to bedrock. Some of these can be inferred from the statistical soil profiles (Appendix A-III). However, due to the variation of topography with location either routine data retrieval or reference to the Soil Survey Manuals (107) may be more helpful.

### 5.2 Shallow Foundations

The design of a foundation unit usually requires that both bearing capacity and settlement be checked.

#### 5.2.a Bearing Capacity

In order to determine the bearing capacity of both cohesive (silty clay, clay) and cohesionless (sand) soils the strength parameters within the depth of influence of the footing are required. The general bearing capacity equation contains the variables of size of the footing, density of the soil, depth of the footing, strength angle, and cohesion of the soil. The density, strength angle,



and cohesion can be estimated either directly from the statistical soil profiles or from the routine retrieval of data.

For many locations in the state, the ground water table is at a shallow depth, viz., a few feet, as shown in Appendix A-I and A-III. Therefore, the soils may often be assumed to be saturated. For cohesive soils the undrained shear strength,  $c_u = \frac{1}{2} q_u$  (unconfined compressive strength), may be used. The  $q_u$  can be obtained either from the appropriate statistical soil profiles, from the regression equation based upon the index properties, as shown in Appendix A-IV, or from routine retrieval of data.

The allowable bearing pressure in sands can be roughly estimated by using the SPT results. For details of the procedures refer to Peck, et al (80). The distribution of SPT versus depth for the area of interest can be obtained either from the statistical soil profiles or from data retrieval.

#### 5.2.b Settlement

The consolidation settlement of cohesive soil is normally calculated using the following soil parameters: the initial void ratio, overburden pressure, compressive index, recompression index, and preconsolidation pressure. The initial void ratio and the overburden pressure are relatively easy to measure or estimate. The other soil parameters can be evaluated by using the regression



equations shown in Appendix A-IV. The preconsolidation pressure can also be roughly estimated from either the statistical soil profiles or from direct data retrieval.

The settlement of cohesionless soil can be estimated by empirically using SPT results. For details refer to Terzaghi and Peck (113), Meyerhof (64) and Peck et al (80).

### 5.3 Pile Foundations

The depth to bedrock can be retrieved from the data bank or estimated by consulting Soil Survey Manuals (107) for the area of interest.

The geometry of the pile, SPT values along the pile, density of the soil, and the strength angle which is used to determine the bearing capacity coefficient are required to predict the ultimate bearing capacity of a single pile in granular soils by the so-called "theoretical method" (64, 74).

The ultimate bearing capacity of a single pile in clay can be estimated by the size of the pile shaft and the adhesion strength, which is correlated with the undrained shear strength of the soil (74).

The density, SPT, strength angle, and the undrained shear strength (half of unconfined compressive strength) can be obtained from either the correlations presented in this work or by data retrieval.





#### 5.4 Cut Slopes

In addition to the information of topography and soil identification, the strength parameters, such as the unconfined compressive strength, cohesion, and strength angle, are required for the analyses of slope stability. A variety of methods can then be applied to the problems using proper sets of strength parameters. For details of these methods refer to Taylor (111), Terzaghi and Peck (113), and Chowdhury (19).

#### 5.5 Excavations and Retaining Structures

The active and passive earth pressure coefficients may be estimated by using the geometry of backfills and retaining structures, and the appropriate strength parameters. For details of analyses refer the NRCC (74), Tschebotarioff (117), and U.S. Navy (118).

#### 5.6 Compaction Requirements for Embankments

The index properties of soils can be used as guidelines for the suitability of excavation or borrow materials. The control compaction parameters, such as maximum dry density and optimum moisture content, are needed for the design and construction of embankments. They can be evaluated for a given soil by using the existing results and regression equations as shown in Appendices A-II, A-III, and A-IV.



### 5.7 Pavements

For the design of roadway pavement the existing results as represented in this work and in the data bank can provide the following information: the topography of the area of interest, soil classification, compaction parameters, i.e., maximum dry density, optimum moisture content, and the soaked CBR values of the subgrade material at either 100% of maximum dry density or 95% maximum dry density. With this information, the corresponding R-value, bearing value, and the modulus of subgrade reaction can be estimated by using the chart developed by the Portland Cement Association (PCA) (83). Providing the traffic information is known, either the flexible (asphalt) or the rigid (concrete) pavement can be designed.

### 5.8 Example 1

The following example is to illustrate the uses of the data bank for a typical highway project.

#### 5.8.1 Project Identification

There is to be a new route along the old S.R. 912 located from East Chicago, Lake County, Indiana to Chicago, Illinois in the area of R9W and T37N. The geotechnical works involve: one bridge, with five spans ranging in length from 90 ft. to 100 ft., one embankment of 30 ft. height, and 5 retaining walls to be constructed along the route. The length of the highway is approximately 3.5 miles. Ten cuts are also along the route. It is desirable to have the



geotechnical information in this area for preliminary considerations.

#### 5.8.2 General Scope of Procedures

The use of the Indiana geotechnical data bank aids in the preliminary phases of design and construction of the project. To accomplish this purpose, the investigation is divided in the following parts:

- (1) determination of the soil types and topography within the area;
- (2) determination of the strength and other engineering characteristics of these soils;
- (3) recommendations to aid the subsurface investigation program;
- (4) recommendations to aid design and construction of the proposed highway.

#### 5.8.3 Determination of the Soil Types

Physiographically, the site is located within the Calumet Lacustrine Plain of the Northern Lake and Moraine Region. The soil association is Oakville-Plainfield-Adrian. The distributions of ground elevation and the water depth with relation to ground elevation were examined. It was found that the mean of the ground elevations was 588 ft., its s.d. was 3.87 ft.; the median was 589 ft. and I.R. was 3.55 ft. Therefore, no appreciable topographic relief was found. The ground water elevation was rather level with median of 6.20 ft. and I.R. of 2.5 ft. According to



the geological maps prepared by the Indiana Geological Survey, the bedrock (limestone) is generally at depths ranging between 100 and 150 ft.

The distributions of textures of soils versus depth for each section along the route were then examined. The subsurface investigation revealed a relatively uniform soil profile. Beneath the ground surface there existed a sandy deposit approximately 32 feet thick with occasional silty sand deposits and gravelly channels at random locations. Surface drainage at this area was good due to the granular nature of the soils.

Beneath the sand, starting at approximately El 557 was a stiff plastic clay. This clay extended approximately 55 ft. (to El 502) and was underlain by a much stiffer and less plastic clay (hard pan) (Figures 5-1 to 5-4).

#### 5.8.4 Determination of the Engineering and Strength Characteristics

The data search program was next directed toward the determination of the engineering and strength characteristics of the various soils encountered in this project. Results are illustrated in Figures 5-1 to 5-4 and tabulated in Table 5-1. Further details of the computer programs have been included in the Appendix B-2.

The results of consolidation tests for clay samples were examined. No substantial preconsolidation is apparent. The clay exhibits a decrease in plasticity with depth, with





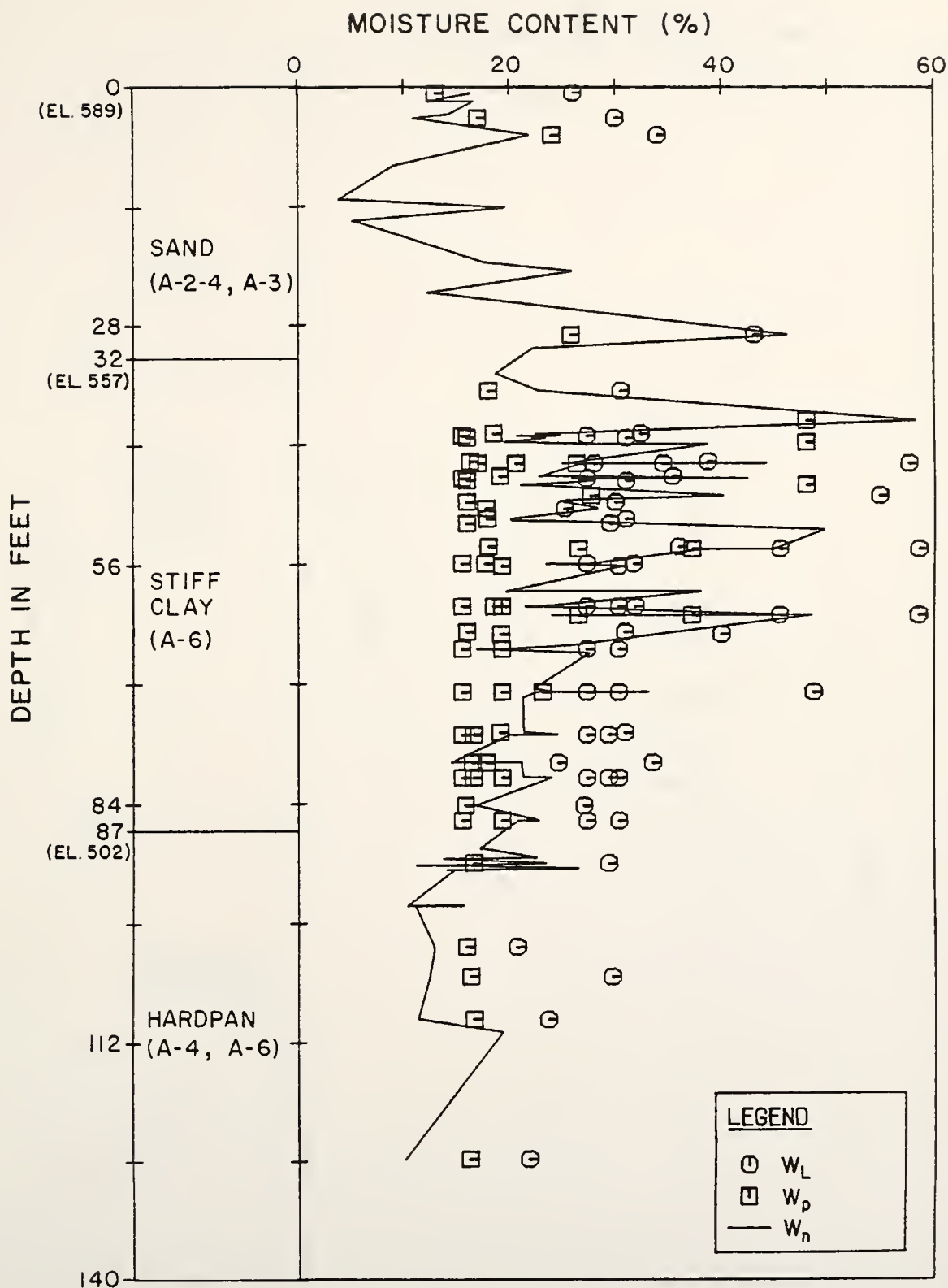


Figure 5-1 Water Content vs Depth, East Chicago  
(Data from Indiana Geotechnical Data Bank)



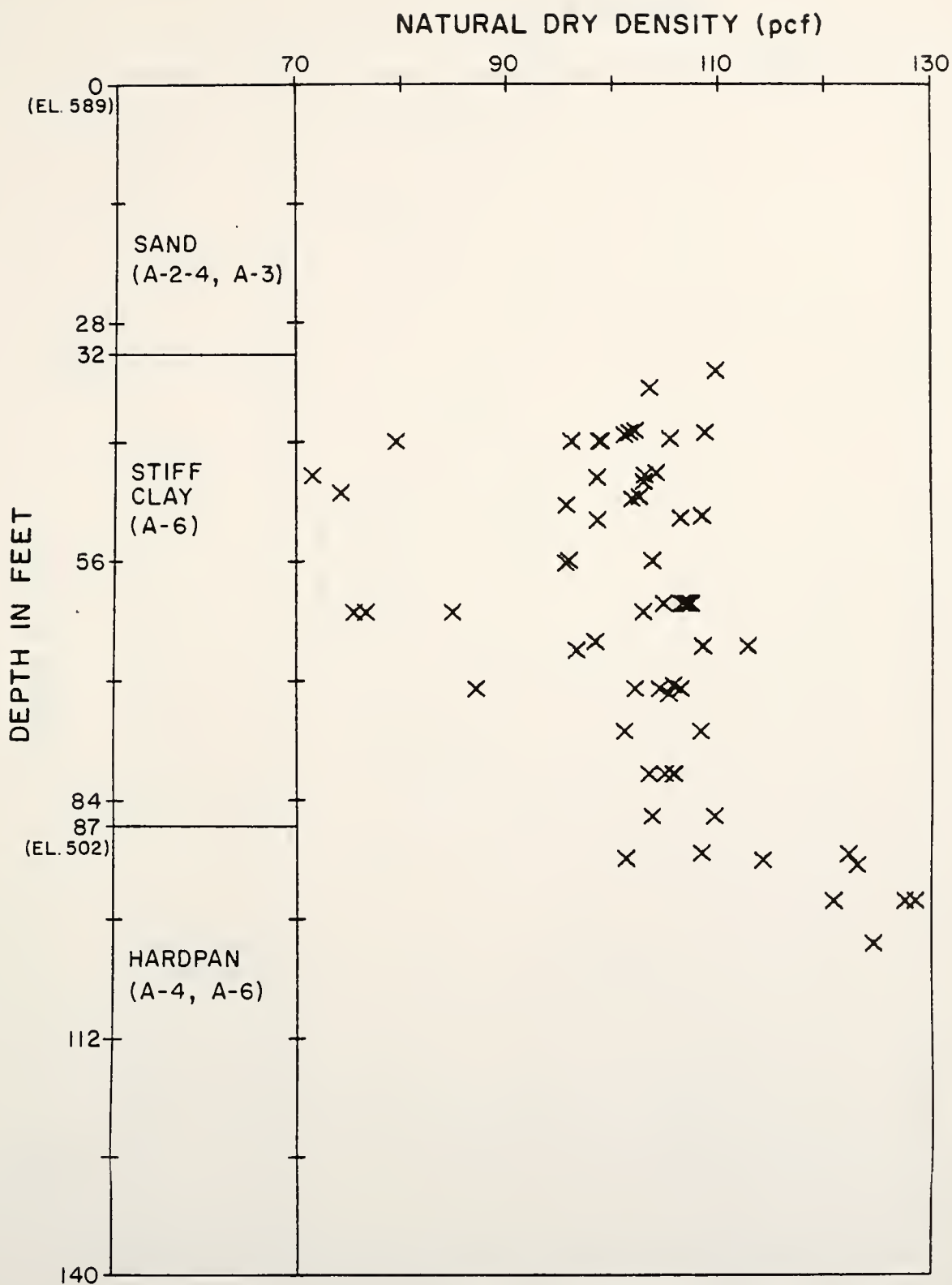


Figure 5-2 Natural Dry Density vs Depth, East Chicago, (Data from Indiana Geotechnical Data Bank)



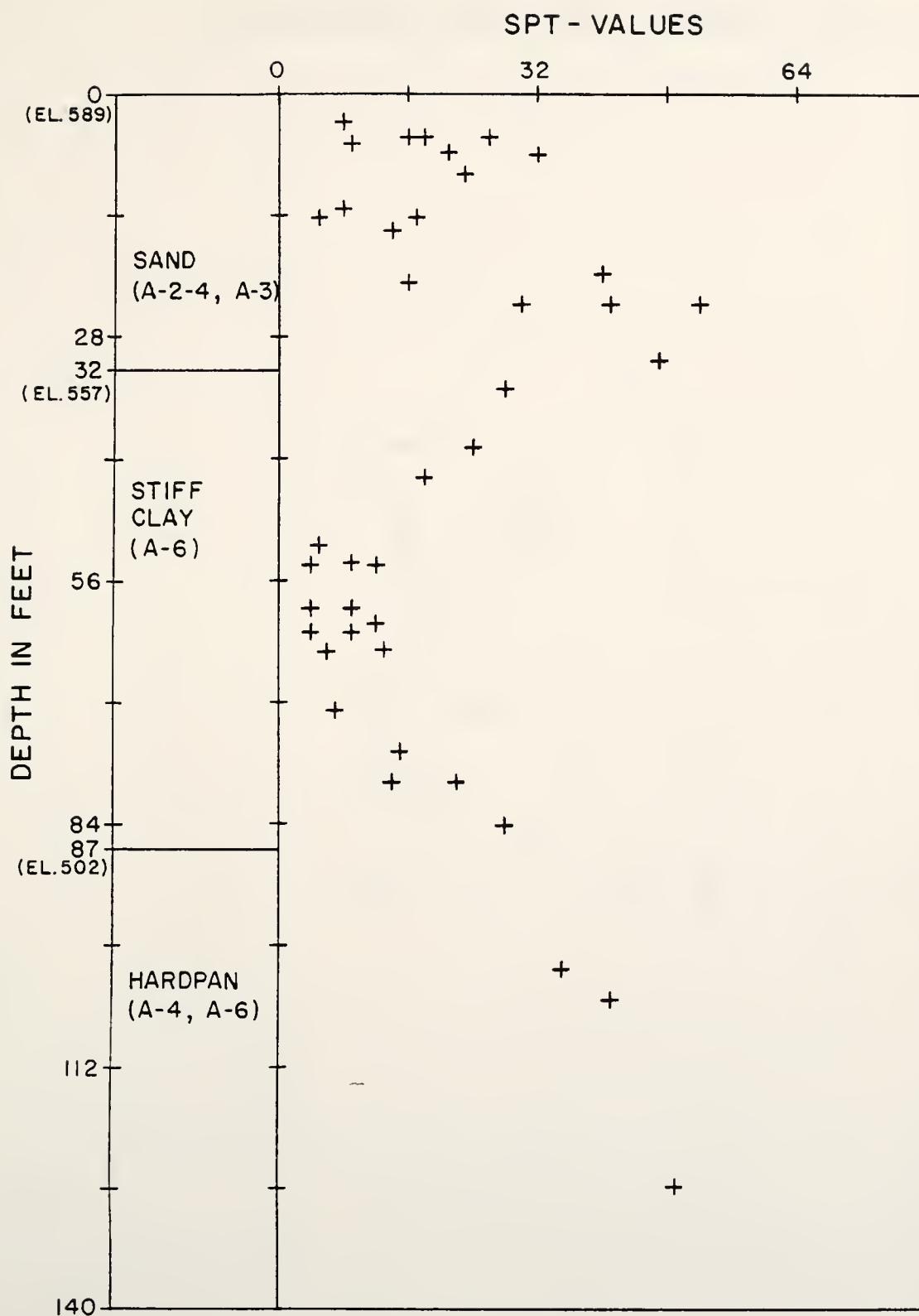


Figure 5-3 SPT vs Depth, East Chicago, (Data from Indiana Geotechnical Data Bank)



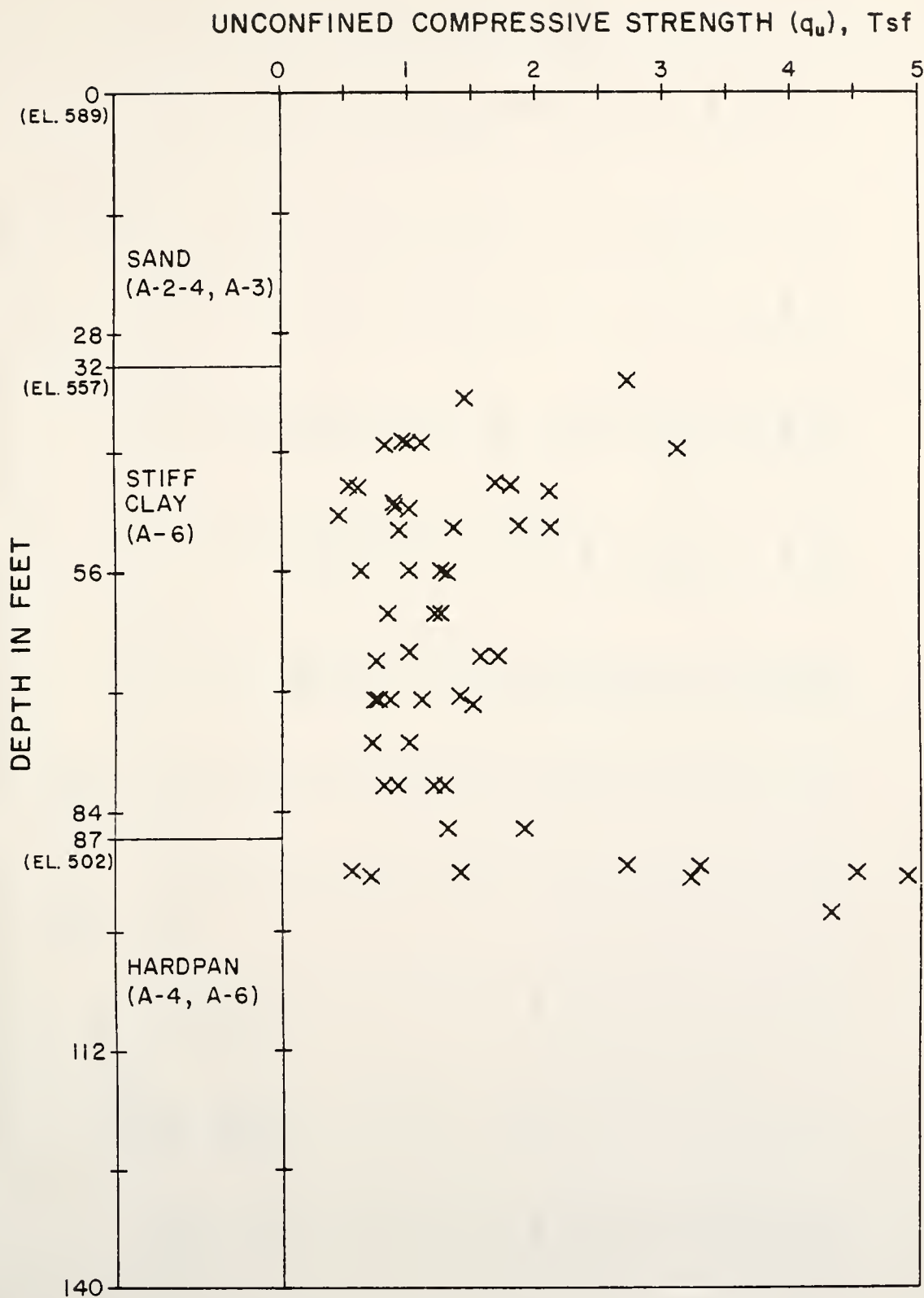


Figure 5-4 Unconfined Compressive Strength vs Depth, East Chicago (Data from Indiana Geotechnical Data Bank)





Table 5-1. Engineering and Strength Characteristics of East Chicago Soils

Depth	G	$\rho_d$ max	OMC	CBR		$e_o$	$C_c$	$C_r$	$c_v$	Type	c	$\phi$
				SO1	SO2							
2.00	2.700	100.7	16.8	31.5	19.2	-0	-0	-0	-0	-0	-0	-0
3.25	-0	99.5	15.0	-0	-0	-0	-0	-0	-0	-0	-0	-0
3.50	-0	102.8	13.7	-0	-0	-0	-0	-0	-0	-0	-0	-0
35.75	-0	-0	-0	-0	-0	.569	.144	.030	-0	-0	-0	-0
40.75	-0	-0	-0	-0	-0	.572	.149	.030	-0	-0	-0	-0
42.00	-0	-0	-0	-0	-0	-0	-0	-0	-0	3.	.05	24.5
42.00	-0	-0	-0	-0	-0	-0	-0	-0	-0	3.	.05	24.5
42.00	-0	-0	-0	-0	-0	-0	-0	-0	-0	3.	.05	24.5
44.00	2.710	-0	-0	-0	-0	.704	.195	.037	1.47	-0	-0	-0
46.00	2.740	-0	-0	-0	-0	.659	.199	.024	-0	-0	-0	-0
46.25	2.740	-0	-0	-0	-0	.729	.238	.037	-0	-0	-0	-0
48.00	2.670	-0	-0	-0	-0	1.392	.590	.130	-0	-0	-0	-0
50.75	-0	-0	-0	-0	-0	.573	.151	.030	-0	-0	-0	-0
51.25	2.740	-0	-0	-0	-0	.739	.220	.037	-0	-0	-0	-0
54.00	2.710	-0	-0	-0	-0	.736	.170	.029	1.47	-0	-0	-0
56.00	2.760	-0	-0	-0	-0	.706	.190	.050	4.26	-0	-0	-0
56.25	2.740	-0	-0	-0	-0	.785	.244	.077	-0	-0	-0	-0



Table 5-1. (continued)

Depth	G	$\rho_d$ max	CBR		$e_o$	$C_c$	$C_r$	$c_v$	Type	c	$\phi$
			OMC	S01	S02						
62.00	2.790	-0	-0	-0	-0	.693	.175	.030	-0	-0	-0
62.00	2.698	-0	-0	-0	-0	1.240	.280	.035	-0	3.	28.5
62.00	2.690	-0	-0	-0	-0	1.240	.280	.035	-0	3.	28.5
62.00	2.690	-0	-0	-0	-0	1.240	.280	.035	-0	3.	28.5
64.00	2.720	-0	-0	-0	-0	.630	.210	.018	1.47	-0	-0
71.00	2.740	-0	-0	-0	-0	1.115	.380	.130	-0	-0	-0
76.00	2.740	-0	-0	-0	-0	.690	.173	.099	-0	-0	-0

Note: 1. (-0) Unknown.

2. (Type 3) Consolidated undrained triaxial test - saturated.

3. Depth - depth to ground surface in ft.

G - specific gravity.

 $\rho_{dmax}$  - maximum dry density (Pcf).

OMC - optimum moisture content (%).

CBR S01 - CBR soaked 100%  $\rho_d$  max (%).CBR S02 - CBR soaked 95%  $\rho_d$  max (%). $e_o$  - initial void ratio $C_c$  - compression index. $C_r$  - recompression index. $c_v$  - coefficient of consolidation  
(ft<sup>2</sup>/mth).

c - cohesion (Tsf)

 $\phi$  - strength angle (degree)



a sharp drop in plasticity at approximately elevation 502 feet (Figure 5-1).

### 5.8.5 Recommendations for Preliminary Design

5.8.5.a Highway Embankment. For the areas where the highway or access ramps will be on fill, standard design and construction procedures, which include compaction requirements, topsoil stripping, and undercutting existing loose fill, are considered satisfactory. It is recommended that standard embankment slopes be used for the project. The embankment slopes will not be steeper than 3 horizontal to 1 vertical (except at end bents where the slopes will be 2 horizontal to 1 vertical).

In some areas, the fill may be retained by retaining walls. The recommendations with respect to design of the retaining walls are discussed in Section 5.8.5.d.

The values of compaction parameters listed in Table 5-1 are recommended for preliminary design. It is predicted that  $\rho_{dmax} = 101$  pcf, and OMC = 15%.

5.8.5.b Settlement of Embankments. Due to the lack of apparent preconsolidation of the foundation clay layer, settlement analysis should be performed on the assumption that it is normally consolidated.

It is recommended that the 55 ft. thick compressible stratum from 32 ft. to 87 ft. be divided into three layers:  $e_o = 0.680$ ,  $C_c = 0.197$ ,  $C_r = 0.031$ , and  $c_v = 1.47 \text{ ft}^2/\text{mth}$  for the layer from 32 ft. to 50 ft.;  $e_o = 0.785$ ,  $C_c =$



0.244,  $C_r = 0.077$ , and  $c_v = 1.47 \text{ ft}^2/\text{mth}$  for the layer from 50 ft. to 68 ft.; and  $e_o = 0.690$ ,  $C_c = 0.173$ ,  $C_r = 0.099$ , and  $c_v = 1.47 \text{ ft}^2/\text{mth}$  for the layer from 68 ft. to 87 ft.

5.8.5.c Bridge Foundations. Both possibilities of using shallow spread footings and deep foundations should be studied. If excessive total settlement and/or intolerable differential settlement are found for shallow spread footings, the deep foundations are recommended. Consideration of scour may also require deep foundations. Piles bearing in the hardpan encountered at about El 492 are recommended. If the end bent of the bridge foundation is also part of the retaining wall supporting the fill, the piles should be sleeved at least down to El 502 to minimize downdrag from negative skin friction resulting from the fill settlements. If the shallow spread footings are applicable, the base should be located at a depth at least 3.5 ft. below final grade for frost protection.

The information in Figures 5-1 to 5-4 and Table 5-1 serves the purpose of preliminary design. The strength angle is predicted to be  $24^\circ$  and the strength intercept is 100 psf.

5.8.5.d Retaining Walls. It is not known what soils will be used for backfill. However, it is assumed that the backfill soils will be clean granular materials. The base of the retaining wall should be located at a depth of at least 3.5 ft. below final grade for frost protection. The





base strength angle is predicted to be  $24^{\circ}$  and the cohesion is 100 psf.

5.8.5.e Ground Water. The ground water elevation appeared to be deep enough to permit footing bottoms to be located below frost depth (3.5 feet) without going below the water table. If the ground water rises (seasonal variation), the footing can be raised to avoid dewatering if an earth berm is placed for the minimum 3.5 ft. of frost cover. However, if the footing must bear below the water table for structural reasons, suitable footing drains must be employed.

5.8.5.f Piles, Corrosion. All pH tests indicated that the soils in this area are not corrosive to concrete and steel.

5.8.5.g Others. The CBR values in Table 5-1 seems high. It is recommended that more extensive CBR testing be performed for subgrade design. If piles are to be used for bridge piers, it is recommended that several pile load tests at various locations along the bridge be performed to determine the suitable pile capacities, as well as the depths to which piles must be driven to attain the desired capacity.

#### 5.8.6 Recommendations for

##### Subsurface Investigation Program

The data bank is not able to precisely locate the position in which the samples have been taken in the previous projects. The smallest location unit is the section (one square mile in area). Therefore, only general



recommendations can be made by using data bank as follows.

5.8.6.a Recommendations by Using Data Bank

1. The standard drilling equipment for making borings is assumed to be used for this project. The standard penetration test is recommended for sand and clay. The Shelby tube is recommended for sampling clay.
2. The depths of the standard penetration tests for the designs of bridge and wall foundations are suggested to be 150 ft. and 32 ft. respectively. Eleven standard penetration tests are required as a minimum. Four intact samples at the depth of 16 ft. beneath ground surface at each bridge pier site and at the depth of 10 ft. beneath ground surface at each retaining wall site are recommended.
3. Shelby tubes are to be used to obtain undisturbed clay samples at the site of embankment at the depths of, approximately 41 ft., 59 ft. and 78 ft. The depth of the boring is 145 ft. (hard pan). Five samples are collected for each specific depth.
4. The pH tests are recommended to be performed on samples at the depths of, approximately, 16 ft., 60 ft. and 100 ft. at each bridge pier site.

5.8.6.b Other Recommendations. Indirect recommendations from other sources, such as the local geological information, the "Requirements for Roadway Soil Survey" by Indiana State Highway Commission, as quoted in McKittrick (63), etc., are as follows:



1. The borings are located alternatively right and left on the roadway centerline at 300 ft. spacings. Therefore, 62 borings are required along the highway. The boring depths are 6 ft. or two thirds the height of the fill (whichever is greater). Hand borings and truck mounted borings with split spoon sampling are recommended.
2. At least one boring should encounter rock, cores should be obtained for a depth of 5 to 10 ft. to make sure that sound bedrock has been reached. If there is evidence of solution channels or deep weathering, the cores should be continued into sound rock. The depth of rock core boring is estimated to be 160 ft.
3. Where possible, ground water observation should be made at the time the borings are completed and twenty-four hours afterwards.
4. Routine classification tests, such as grain size distribution and Atterberg limits, should be conducted on samples of each stratum encountered on the project.
5. Consolidation and unconsolidated undrained triaxial tests are performed on Shelby tube samples for the analyses of settlement and bearing capacity of the embankment and other structures.
6. Compaction and CBR tests are performed in cut areas. If the local materials are not used as fill, these tests will not be necessary, but will still be run on borrow materials. Ten cut areas are assumed along the highway.



#### 5.8.6.c Conclusions

1. The data bank can give a general impression of the subsurface materials and permit prediction of efficient drilling and sampling equipment.
2. Using the data bank the number, type and depths of samples can be better estimated.
3. The expected values from testing of extracted samples can be obtained from the data bank and used in quality control for the actual experimental measurements.

5.8.6.d Summary. The quantity estimates of the recommended subsurface investigation program are shown in Table 5-2.

### 5.9 Example 2

The following example illustrates how state, county, and city engineers may be supplied with presumptive CBR data for preliminary pavement designs through reference to the Indiana data bank.

#### 5.9.a Project Identification

There is to be a new route located in the north of the city of Evansville, Vanderburg County. The city engineer needs CBR data for preliminary pavement design.

#### 5.9.b General Scope of Procedures

The compiled results of the Indiana geotechnical data bank aid in the preliminary phase of pavement design. To accomplish this objective, the investigation is divided into





Table 5-2 The quantity estimates of the recommended  
subsurface investigation program

---

Borings and Sampling

	<u>Qty.</u>
Hand borings	<u>372 Lft</u>
Truck mounted borings with split spoon sampling	<u>1205 Lft</u>
Truck mounted rock core borings	<u>160 Lft</u>
3-in undisturbed samples	<u>15 Ea.</u>
Split spoon samples	<u>44 Ea.</u>

Laboratory testings

Grain size distribution	<u>62 Ea.</u>
Liquid limit	<u>62 Ea.</u>
Plastic limit	<u>62 Ea.</u>
Natural moisture content	<u>72 Ea.</u>
Consolidation test	<u>3 Ea.</u>
Unconsolidated undrained triaxial test	<u>14 Ea.</u>
pH test	<u>18 Ea.</u>
OMC test*	<u>10 Ea.</u>
Maximum dry density test*	<u>10 Ea.</u>
CBR test*	<u>10 Ea.</u>
Bridge foundation soil analysis and recommendation	<u>6 Ea.</u>

---

\*If excavation materials are not used as fill, these tests  
will be performed in different quantities.



Table 5-2 (continued)

---

Embankment settlement analysis	<u>1 Ea.</u>
Stability analysis of retaining walls	<u>5 Ea.</u>
Soils report	<u>LS</u>
Resident soils engineers	<u>Ea.</u>

---



two parts:

- (1) determination of the soil types and their corresponding CBR values within the area;
- (2) recommendation of preliminary design CBR values.

#### 5.9.c Determination of the Soil Types and Their Corresponding CBR Values

The soil association was found to be McGary in the Wabash Lowland. From an examination of its statistical soil profile (Table A-III-23, Appendix A) it was found that the texture of the top ten feet was silty clay loam (A-4, A-6, A-7-6). The medians of soaked CBR at 100% maximum dry density (CBR S01) and soaked CBR at 95% maximum dry density (CBR S02) are 6.75 and 3.93 respectively. In order to confirm the results, the tables of remolded soil characteristics within the Wabash Lowland (Table A-II-22 to A-II-24) were then examined. It was found that for A-4 soils the medians of CBR S01 and CBR S02 were 10.00 and 5.60 respectively; for A-6 soils the medians of CBR S01 and CBR S02 were 7.25 and 4.47, respectively; and for A-7-6 soils the medians of CBR S01 and CBR S02 were 5.80 and 4.00, respectively.

#### 5.9.d Recommendation of the Preliminary Design CBR Values

With this information it is recommended that the CBR S01 value be 6.75 and the CBR S02 value be 3.92 as the medians are used.



### 5.10 Example 3

It is desired to predict the generalized line of optimums for Standard AASHTO laboratory compaction for the soils of Indiana.

#### 5.10.a Procedures

A scatter plot of  $\rho_{dmax}$  vs. OMC was first examined. A strong curvilinear trend of decreasing  $\rho_{dmax}$  with increasing OMC was shown. A second degree polynomial model was used for best fit.

#### 5.10.b Results

The equation was found to be

$$\rho_{dmax}(pcf) = 150.667 - 3.016 \text{ OMC} + 0.0333 (\text{OMC})^2,$$

for which  $|R| = 0.906$ , s.d. of est. = 3.691, and No. of cases = 701. The scatter plot with its best-fit line is shown in Figure 5-5. For example, if OMC = 16.00%,  
 $\rho_{dmax} = 110.93 \text{ pcf}.$





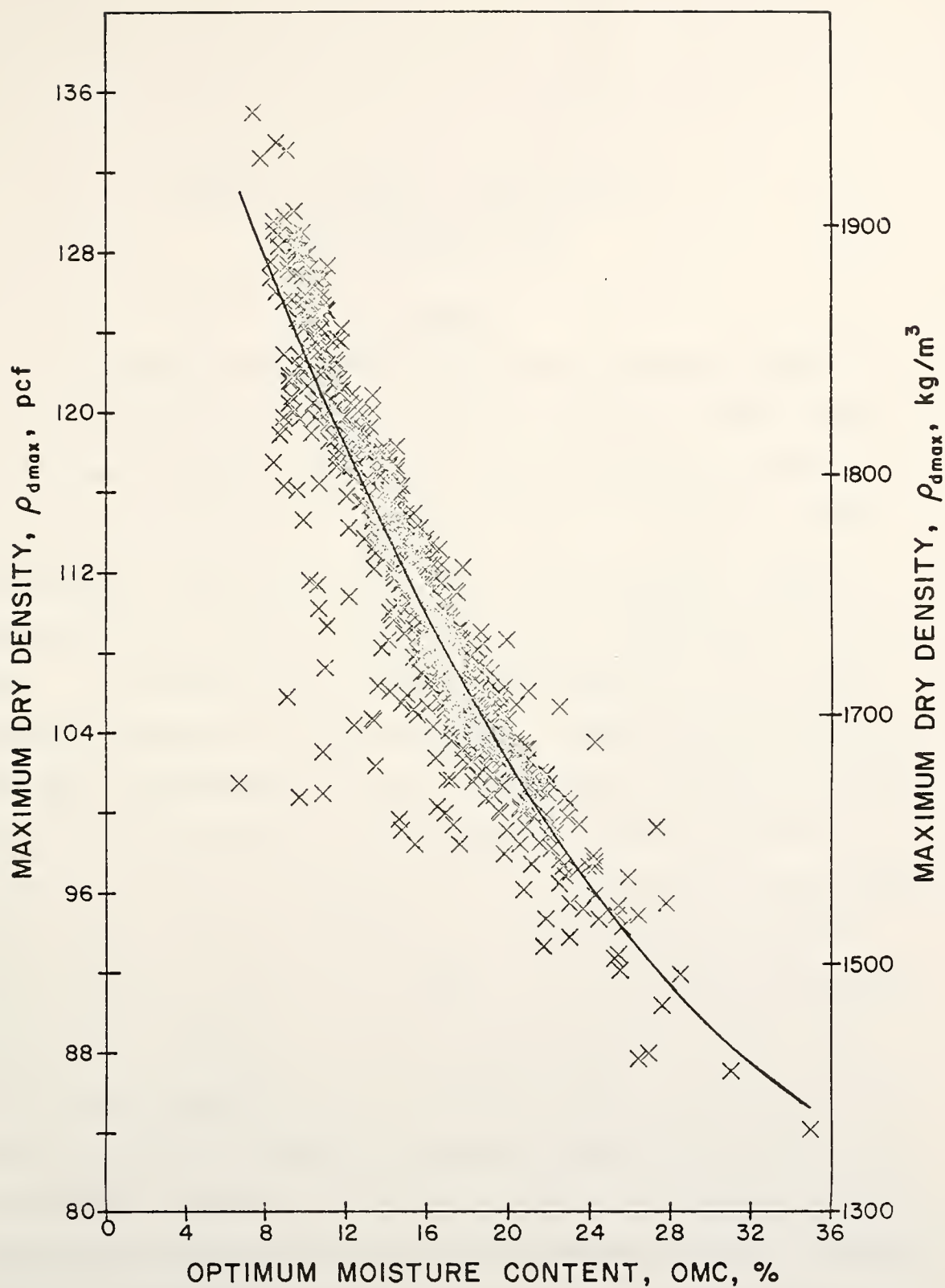


Figure 5-5 Maximum Dry Density vs Optimum Moisture Content, Indiana Soils



## CHAPTER 6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

### 6.1 Summary

A computerized, data storage and retrieval system has been developed for the State of Indiana. Both conventional and nonparametric statistical methods have been employed in the analysis of these data. The studies on the topographic characteristics versus physiographic region (Appendix A-1) were based on a one-way classification layout. The results give a general impression of the topographic features of a physiographic region and can be used to make comparisons of overall topographic features between any two such regions. The methods can also be applied to smaller areas if topographic data are adequately distributed. The studies on the remolded soil characteristics versus physiographic regions and AASHTO classifications (Appendix A-II) were based on a two-way classification layout, and show the relationships on a regional basis. The studies on statistical soil profiles (Appendix A-III) were based on a factorial experiment. The results show the general subsoil conditions qualitatively and the estimates of soil characteristics quantitatively with depths for soil associations on a regional basis. Finally, the regression analysis



(Appendix A-IV) shows the functional relationships between design parameters and index properties.

The examples shown in Chapter 5 illustrates specific uses of the geotechnical data bank.

## 6.2 Conclusions

- (a) Topographic features vary with physiographic regions. (Section 4.2.a)
- (b) The remolded soil characteristics can be evaluated and contrasted between physiographic regions but also within AASHTO classifications. (Appendix A-II.)
- (c) The distributional data confirm that the more plastic\* soils have lower maximum dry density and higher optimum moisture content values (Section 4.2.b).
- (d) The soil association is the most refined unit for grouping soils to generate soil profiles at the present stage of study (Section 4.2.d).
- (e) Statistical regressions show that as the liquid limit or plastic limit of a soil increases, the optimum moisture content increases but maximum dry density decreases. This confirms the findings of earlier authors (Section 4.3.a and Appendix A-IV).
- (f) A unique relationship exists between optimum moisture content and maximum dry density which confirms the findings of earlier authors (Section 4.3.a and Appendix A-IV).

---

\* Soils with higher values of the plasticity index.



- (g) The maximum dry density is not significantly influenced by geological factors but the optimum moisture content is (Appendix A-IV).
- (h) The CBR value is a function of plasticity characteristics and a correlation between the CBR value at 100% maximum dry density and the CBR value at 95% maximum dry density exists which confirm the findings of earlier authors (Section 4.3.a and Appendix A-IV).
- (i) Compression index ( $C_c$ ) is a function of natural moisture content, initial void ratio, and liquid limit and is significantly influenced by geological factors, which confirms the findings of earlier authors. With 95% confidence the recompression index ( $C_r$ ) lies in the range of, approximately,  $\frac{1}{6.5} C_c$  and  $\frac{1}{8} C_c$  for Indiana soils (Section 4.3.b and Appendix A-IV).
- (j) The preconsolidation pressure ( $p_c$ ) is a function of natural water content, initial void ratio, and natural dry density and is significantly influenced by geological factors. But the scatter in the data is large (Section 4.3.b and Appendix A-IV).
- (k) No definite correlation of strength angle and cohesion intercept versus plasticity characteristics was found for Indiana soils (Section 4.3.c).





- (l) Non-parametric statistical methods are preferred, as opposed to conventional statistical methods, for data analyses (Section 3.3.1.a).
- (m) The data bank is valuable for making recommendations for preliminary design of geotechnical works (Chapter 5).
- (n) The physiographic region, engineering soil classification, soil association, and a combination of them were used as grouping units. The interquartile ranges (IR's) as shown in Appendix A-II and A-IV for most soil characteristics are small and tolerable. In other words, a good homogeneity of soil characteristics is evidenced with these groupings.
- (o) It is emphasized that the data bank is not proposed as a substitute for fuller site investigation, sampling and testing, but as a framework against which various test results can be judged for their consistency and reliability.

### 6.3 Recommendations for Future Research

- (a) In addition to displaying the distribution of each data item based on physiographic region, or engineering soil classification, or soil association or a combination of them, cluster analysis and principle components analysis (23, 28, 68) are recommended for future research. These combine



the characteristics of each sample into a general geotechnical character for the soil and group the samples into similar geotechnical zones. The proper selection of soil characteristics will play an important role in correctly identifying geotechnical zones.

- (b) The feasibility of adding information to the existing data file should be studied.
- (c) The feasibility of establishing a computerized geotechnical information library, in which data sources can be searched and attached to the data bank, should be studied. The data sources include more subsurface investigation reports, geological surveys, agricultural soil surveys, and published relationships among soil characteristics in the State of Indiana.
- (d) The feasibility of establishing a permanent facility to operate and maintain the data bank and to provide services to potential users, such as private consultants and contractors should be studied. This facility would keep the data file current and would make revisions of existing correlations and regression equations among soil characteristics.



## BIBLIOGRAPHY



## BIBLIOGRAPHY

1. American Hoist and Derrick Company (1977), "Handbook of Soil Compactionology", St. Paul, Minnesota, 53 pp.
2. Anderson, F. L. and McLean, R. A. (1974), "Design of Experiments", Marcel Dekker, Inc., New York, 418 pp.
3. Anscombe, F. J. (1973), "Graphs in Statistical Analysis", American Statistician, Vol. 27, pp. 17-21.
4. Asphalt Institute (1965), "Thickness Design - Asphalt Pavement Structures for Highways and Streets", MS-1, December, 95 pp.
5. Asphalt Institute (1970), "Thickness Design - Full-Depth Asphalt Pavement Structures for Highways and Streets", MS-1, August, 80 pp.
6. Azzouz, A. S., Krizek, R. J., and Corotis, R. B. (1976), "Regression Analysis of Soil Compressibility", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 16, No. 2, pp. 19-29.
7. Bazaraa, A. R. S. (1967), "Use of the SPT for estimating settlements of Shallow Foundation on Sand", Ph.D. Thesis, University of Illinois, Urbana, Illinois, 381 pp.
8. Belcher, D. J., Gregg, L. E., and Woods, K. B. (1943), "The Foundation, Distribution, and Engineering Characteristics of Soils", Joint Highway Research Project, the State Highway Commission of Indiana - The Engineering Bulletin No. 87, Purdue University, January, 389 pp.
9. Bishop, A. W. and Eldin, G. (1950), "Undrained Triaxial Tests on Saturated Sands and Their Significance in the General Theory of Shear Strength", Geotechnique, Vol. 2, pp. 13-32.
10. Bjerrum, L. and Simons, N. (1960), "Comparison of Shear Strength Characteristics of Normally Consolidated





- Clays", Proceedings of ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, pp. 711-726.
11. Bjerrum, L. (1972) "Embankments on Soft Ground", State-of-the-Art Report, Proc. ASCE Spec. Conf. on Performance of Earth and Earth-Supported Structures, Lafayette, Vol. 11, pp. 1-54.
  12. Black, W. P. M. (1962), "A Method of Estimating CBR of Cohesive Soil from Plasticity Data", Geotechnique, Vol. 12, No. 4, pp. 271-282.
  13. Black, W. P. M. and Lister, N. W. (1979), "The Strength of Clay Fill Subgrades: its Predictions in relation to Road Performance", Transport and Road Research Laboratory, Berkshire, England, 30 pp.
  14. Bradley, J. V. (1968), "Distribution-Free Statistical Tests", Prentice-Hall, N.J., 388 pp.
  15. Caquot, A. and Kerisel, J. (1953), "Courbes de glissement sous la pointe des pieux", Journées de la Mécanique des Sols. Ann. I.T.B.T.P., pp. 341-342.
  16. Caquot, A. and Kerisel, J. (1966), "Traité de Mécanique des Sols", Gauthier-Villars, Paris, 4th Ed., 506 pp.
  17. Casagrande, A. (1948) "Classification and Identification of Soils", Trans. ASCE, Vol. 13, pp. 901-992.
  18. Chatterjee, S. and Price, B. (1977), "Regression Analysis by Example", John Wiley and Sons, N.Y., 228 pp.
  19. Chowdhury, R. N. (1978), "Slope Analysis", Elsevier Publishing Company, Amsterdam. pp. 78-244.
  20. Cochran, W. G. and Cox, G. M. (1957), "Experimental Designs", 2nd Ed., John-Wiley and Sons, N.Y., 611 pp.
  21. Cozzolino, V. M. (1961), "Statistical Forecasting of Compression Index", Proceedings of the Fifth Int. Conf. on Soil Mech. and Fdn. Eng., Paris, Vol. 1, pp. 51-53.
  22. Crawford, R. A., Thomas, J. B., and Stout, M. Jr. (1972), "Computerized Soil Test Data for Highway Design", Physical Research Section, South Dakota Department of Highways, Pierre, South Dakota, 24 pp.



23. Cubitt, J. M., Andrews, D. E., and Denness, B. (1978), "Automatic Identification and Evaluation of Geotechnical Zones for Till", Bulletin of the Association of Engineering Geologists, Vol. XV, No. 4, pp. 353-374.
24. Data Transformation Corporation, (1978), "Materials and Test Data Information Systems in Highway Construction", Technology Sharing Report, FHWA-TS-78-111, U.S. Department of Transportation, Washington, D.C., 156 pp.
25. Dewan, R. L. (1963), "Empirical Relationship between Wet Density and Moisture Content of Soil under Standard Compaction", Journal of the Indian National Society of Soil Mechanics and Foundation Engineering, Vol. 1, No. 4, October, pp. 311-333.
26. Draper, N. R., and Smith, H., "Applied Regression Analysis", John Wiley and Sons, N.Y., 407 pp.
27. Elnaggar, H. A. and Krizek, R. J. (1970), "Statistical Approximation for Consolidation Settlement", Highway Research Record, No. 313, pp. 87-96.
28. Everitt, B. (1974), "Cluster Analysis", Social Science Research Council, Heinemann Educational Books Ltd, London, 122 pp.
29. Fadum, R. E. (1941), "Observations and Analysis of Building Settlements in Boston", Sc.D. Thesis, Harvard University.
30. Franzmeir, D. P., Sanders, F. W. and Zachary, A. L. (1975), "Key to Soils of Indiana," Department of Agronomy, Purdue University and USDA Soil Conservation Service, February, 26 pp.
31. Gawith, A. H. and Perrin, C. C. (1961), "Development in the Design and Construction of Bituminous Surfaced Pavements in the State of Victoria, Aust.", Int. Conf. Structural Design of Asphalt Pavements, University of Michigan, pp. 897-910.
32. General Soils Maps and Interpretation Tables for the Counties of Indiana (1971), Agriculture Experiment Station/Cooperative Extension Service, Purdue University and Soil Conservation Service, U.S. Department of Agriculture, November.
33. Gilchrist, W. (1976), "Statistical Forecasting", John Wiley and Sons, N.Y., 300 pp.



34. Glossop, R. (1968), "The Rise of Geotechnology and its Influence on Engineering Practice", 8th Rankine Lecture, Institute of Civil Engineers, London.
35. Goldberg, G. D. (1978), "Development of the Computerized Geotechnical Data Bank for the State of Indiana", MSCE Thesis, Purdue University, May. See also Joint Highway Research Project Report No. 76-6, 163 pp.
36. Goldberg, G. D., Lovell, C. W. and Miles, R. D. (1978), "Use the Geotechnical Data Bank!" Low Volume Roads" Second International Conference, Transportation Research Record 702, August, pp. 140-146.
37. Greenstein, J. and Livneh, M. (1975), "A Procedure for Estimating the Design CBR Value of a Dune Sand", Geotechnical Engineering, Journal of South East Asian Society of Soil Engineering, Singapore, pp. 155-166.
38. Hastings, N. A. J. and Peacock, J. B. (1975), "Statistical Distributions", Halsted Press, John Wiley and Sons, N.Y., 129 pp.
39. Highter, W. H. (1969), "Temperature Effects on the Compaction and Strength' Behavior of a Clay", MSCE Thesis, Purdue University, August, 83 pp.
40. Hogg, R. V. and Craig, A. T. (1970), "Introduction to Mathematical Statistics", 3rd Ed., Macmillan Publishing Co., N.Y., 415 pp.
41. Holden, A. (1969), "Engineering Soil Mapping from Airphotos", Ministry of Roads and Traffic, Salisbury, Rhodesia, July.
42. Hollander, M., and Wolfe, D. A. (1973), "Nonparametric Statistical Methods", John Wiley and Sons, N.Y. 503 pp.
43. Hough, B. K. (1957), "Basic Soils Engineering", The Ronald Press Company, N.Y., pp. 114-115.
44. Il'nitskii, K. N. and Shvets, V. B. (1979), "Standardized Characteristics of Eluvial Clay Soils Formed by Weathering of Argillites and Aleurolites", Soil Mechanics and Foundation Engineering, State Committee for Construction, USSR, September, pp. 70-73.
45. Jenny, H. (1941), "Factors of Soil Formation", McGraw-Hill Book Company, N.Y. pp. 16.
46. Johnstone, J. G., Ramarathnam, S., and Richards, D. B. (1962), "The Soils of Eastern Colorado - their origin,





distribution and engineering characteristics", Quarterly of the Colorado School of Mines, Vol. 57, No. 3, Golden, Colorado, 71 pp.

47. Jorgenson, J. L. (1967), "Predicting the California Bearing Ratio from Compaction and Classification Data", Highway Research Board Record 156, pp. 62-63.
48. Kassiff, G., Livneh, M., and Wiseman, G. (1969), Pavements on Expansive Clays", Jerusalem Academic Press, Jerusalem, Israel, pp. 103-117.
49. Kogure, K. and Ohira, Y. (1977), "Statistical Forecasting of Compressibility of Peaty Ground", Canadian Geotechnical Journal, Vol. 14, No. 4, pp. 562-570.
50. Krebs, R. D. and Walker, R. D. (1971), "Highway Materials", McGraw-Hill Book Company, N.Y., 428 pp.
51. Larson, H. J. (1973), "Introduction to the Theory of Statistics", John Wiley and Sons, N.Y. 242 pp.
52. Leonards, G. A. (1962), "Engineering Properties of Soils", Chapter 2 in "Foundation Engineering", edited by G. A. Leonards, McGraw-Hill Book Company, N.Y., pp. 145-156.
53. Lille Regional Laboratory (1971), "Establishment of a Card Index for Borings and Tests", Ministry Public Works, Paris, January.
54. Liu, T. K. (1967), "A Review of Engineering Soil Classification Systems", Highway Research Board Record 156, pp. 1-22.
55. Livneh, M. and Ishai, I. (1975), "Prediction of CBR Values under Covered Areas", Proc. 2nd Australia-New Zealand Conference on Geomechanics, Institute of Engineers, Brisbane, Australia, pp. 109-114.
56. Lo, Y. K. (1977), "Strength-Composition Relationships of Some Selected Hawaiian Soils", M.S. Thesis, Iowa State University, Ames, Iowa. 188 pp.
57. Lovell, C. W., Jr. (1979), Class Notes from A Course on Earth Structures, Purdue University, Lafayette, Indiana.
58. Lundin, S. E., Stephansson, O., and Zetterlund, P. (1973), "Geoteknisk Databank", Rapport R70, National Swedish Building Research Institute, 158 pp.





59. Lyndon, A., Schofield, A. N. (1978), "Centrifugal Model Tests of the Lodalén Landslide", Canadian Geotechnical Journal, Vol. 15, No. 1, February. pp. 1-13.
60. Maeguchi, S., Sakai, K., and Oyamada, H. (1965), "On Soil Investigations of the Foundation of Kuchiro Power Station", Soil and Foundation, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 13, No. 8, pp. 3-9.
61. May, R. W. and Thomson, S. (1978), "The Geology and Geotechnical Properties of Till and Related Deposits in the Edmonton, Alberta, Area", Canadian Geotechnical Journal, Vol. 15, No. 3, pp. 362-370.
62. Map of the Soil Associations of Indiana (1977), Co-operative Extension Service, Purdue University in Cooperation with the State Soil and Water Conservation Committee, Indiana Department of Natural Resources, and the Soil Conservation Service, U.S.D.A.
63. McKittrick, D. P. (1966), "Subsurface Investigation for Indiana Highways", MSCE Thesis, Purdue University, West Lafayette, Indiana, September. See also Joint Highway Research Project Report No. 20, 176 pp.
64. Meyerhof, G. G. (1956), "Penetration Tests and Bearing Capacity of Cohesionless Soils", Journal of Soil Mech. and Foundation Division, ASCE, No. 82: SM1, Paper No. 866.
65. Meyerhof, G. G. (1957), "The Ultimate Bearing Capacity of Foundations on Slopes", Proc. 4th Int. Conf. on Soil Mech. and Foundation Eng., Vol. I, London, pp. 384-386.
66. Meyerhof, G. G. (1965), "Shallow Foundation", Journal of Soil Mech. and Foundation Division, ASCE, No. 91: SM2, pp. 21-31.
67. Mood, A. M., and Graybill, F. A. (1963), "Introduction to the Theory of Statistics", 2nd Ed., McGraw-Hill Book Company, N.Y., 408 pp.
68. Moran, Proctor, Mueser, and Rutledge (1958), "Study of Deep Soil Stabilization by Vertical Sand Drains", Bureau of Yards and Docks, Department of the Navy, Contract No. NOY-88812.
69. Morgenstern, N. R. and Thomson, S. (1971), "Comparative Observations on the Use of the Pitcher Sampler in Stiff Clay", ASTM Special Publication 483, pp. 180-191.



70. Morrison, D. F. (1976), "Multivariate Statistical Methods", 2nd Ed., McGraw-Hill Book Company, N.Y., pp. 266-345.
71. Narayana Murty, P. L. (1965), "Relationship between the Plasticity Characteristics and Compaction of Soils", Journal of the Indian National Society of Soil Mech. and Foundation Engineering, Vol. 4, No. 4, pp. 455-463.
72. Narain, J. and Ramanathan, T. S. (1970), "Variation of Atterberg Limits in relation to Strength Properties of a highly Plastic Clay", Journal of the Indian National Society of Soil Mech. and Foundation Engineering, Vol. 9, No. 2, pp. 117-128.
73. National Research Council of Canada (1969), "Muskeg Engineering Handbook", Muskeg Subcommittee, Associate Committee on Geotechnical Research, University of Toronto Press, Toronto, Ont. 297 pp.
74. National Research Council of Canada (1975), "Canadian Manual on Foundation Engineering", draft, pp. 24.
75. Nie, N. H., Hull, C. H., Jenkins, J. G., Steinbrenner, K. and Bent, D. H. (1975), "Statistical Package for Social Sciences", 2nd Ed., McGraw-Hill Book Company, N.Y. 675 pp.
76. Nishida, Y. (1965), "A Brief Note on Compression Index of Soil", Journal of the Soil Mech. and Foundation Division, ASCE, Vol. 82, No. SM3, pp. 1027-1 -- 1027-14.
77. Ohsaki, Y. (1962), "Geotechnical Properties of Tokyo Subsoils", Soil and Foundation, the Japanese Society of Soil Mech. and Foundation Engineering, Vol. 11, No. 2, pp. 17-34.
78. Osterberg, J. O. (1972), Personal communication between Osterberg and Azzouz et al.
79. Peck, R. B. and Reed, W. C. (1954), "Engineering Properties of Chicago Subsoils", Engineering Experiment Station, University of Illinois, Bulletin 423.
80. Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974), "Foundation Engineering", 2nd Ed., John Wiley and Sons, N.Y. pp. 20, pp. 114, pp. 307-330.
81. Perloff, W. H. and Baron, W. (1976), "Soil Mechanics: Principles and Applications", The Ronald Press Company, N.Y. 745 pp.



82. Peters, N. and Lamb, K. N. (1979), "Experiences with Alluvial Foundations for Earth Dams in the Prairie Provinces", Canadian Geotechnical Journal, Vol. 16, No. 2. pp. 255-271.
83. Portland Cement Association (1966), "Thickness Design for Concrete Pavements", Skokie, Illinois. 72 pp.
84. Proctor, R. R. (1933), "Fundamental Principles of Soil Compaction", Engineering News-Record, Vol. 3, August 31, pp. 245-248.
85. Raghavan, N. and Bhaskaran, V. (1973), "Evaluation of Compression Index of Tamil Nadu Clays from their Engineering Characteristics", Indian Geotechnical Journal, Vol. 3, No. 3. pp. 232-239.
86. Ramiah, B. K., Viswanath, C., and Krishnamurthy, H. V. (1970), "Interrelationship of Compaction and Index Properties", Proc. 2nd Southeast Asian Conf. on Soil Engineering, Singapore, pp. 577-587.
87. Redon, J. P. (1972), "Computerized Inventory of Soil Profiles throughout Algeria", National Laboratory of Public Works and Buildings, Algeria, September.
88. Rivard, P. J. and Goodwin, T. E. (1978), "Geotechnical Characteristics of Compacted Clays for Earth Embankments in the Prairie Provinces", Canadian Geotechnical Journal, Vol. 15, No. 3, pp. 391-401.
89. Road Research Laboratory (1952), "Soil Mechanics for Road Engineers", London.
90. Robinson, P. J. M. and Lewis, T. (1958), "A Rapid Method of Determining in situ CBR Values", Geotechnique, Vol. 8, No. 2, pp. 72.
91. Roscoe, K. H., Schofield, A. N., and Wroth, C. P. (1958), "On the Yielding of Soils", Geotechnique, Vol. 8, No. 1, pp. 22-52.
92. Ruhe, R. V. (1975), "Geomorphology", Houghton Mifflin Company, Boston. 246 pp.
93. Rutledge, P. C. (1939), "Compression Characteristics of Clays and Application to Settlement Analysis", ScD. Thesis, Harvard University.
94. Sahasi, K. M. S. (1974), "Correlation of Unconfined Compressive Strength of Soil with Size of the Test Specimen", Indian Geotechnical Journal, Vol. 4, No. 4, pp. 362-368.





95. Sanglerat, G. (1972), "The Penetrometer and Soil Exploration", Elsevier Publishing Company, Amsterdam, 464 pp.
96. Schmertmann, J. H. (1955), "The Undisturbed Consolidation Behavior of Clay", Trans. ASCE, Vol. 120, pp. 1201.
97. Scott, C. R. (1969), "An Introduction to Soil Mechanics and Foundations", Applied Science Publishers, Ltd., London. 316 pp.
98. Sengupta, D. P. and Juneja, P. L. (1965) "Prediction of Compression Index from Soil Properties", Indian Journal of Technology, Vol. 3, No. 10. pp. 328-331.
99. Sharma, R. C. and Sharma, S. K. (1964), "Principles and Practice of Highway Engineering", Asia Publishing House, New Delhi.
100. Sherif, M. A. and Burrous, C. M. (1969), "Temperature Effects on the Unconfined Shear Strength of Saturated, Cohesive Soil", Highway Research Board Special Report 103, pp. 267-272.
101. Siegel, S. (1956), "Nonparametric Statistics for the Behavioral Sciences", McGraw-Hill Book Company, N.Y. 312 pp.
102. Sisiliano, W. J. (1970), "A Regional Approach to Highway Soils Considerations in Indiana", Joint Highway Research Project Report No. 18, Purdue University, September. 223 pp.
103. Skempton, A. W. (1944), "Notes on the Compressibility of Clays", Quarterly Journal of the Geological Society of London, Vol. 100, pp. 119-135.
104. Skempton, A. W. and Northey, R. D. (1953), "The Sensitivity of Clays", Geotechnique, Vol. 3, No. 1, pp. 30-53.
105. Skempton, A. W. (1957), Discussion on "The Planning and Design of the New Hong Kong Airport" by Grace and Henry, Proc. Inst. Civil Eng., Vol. 7, 1957, pp. 305.
106. Snedecor, G. W. and Cochran, W. G. (1967), "Statistical Methods", 6th Ed., The Iowa State University Press, Ames, Iowa. 593 pp.
107. Soil Survey Manuals (1958 - present), United States Department of Agriculture, Soil Conservation Service.





108. Sowers, G. B. and Sowers, G. F. (1970), "Introductory Soil Mechanics and Foundations", The Macmillan Company, N.Y. pp. 102.
109. Sprading, D. (1976), "A Soils Data System for Kentucky", Research Report 441, Division of Research, Bureau of Highways, Kentucky Department of Transportation. 58 pp.
110. Stephenson, H. K., Karrah, J. B., and Koplon, N.A. (1967), "Compacted Soil Strength Estimated from Grain Size Distribution and Soil Binder Analysis", Highway Research Record, No. 190, pp. 1-7.
111. Taylor, D. W. (1948), "Fundamentals of Soil Mechanics", John Wiley and Sons, N.Y. pp. 406-479.
112. Terzaghi, K. (1943), "Theoretical Soil Mechanics", John Wiley and Sons, N.Y. 510 pp.
113. Terzaghi, K. and Peck, R. B. (1967), "Soil Mechanics in Engineering Practice", 2nd Ed., John Wiley and Sons, N.Y. pp. 73, pp. 112, pp. 193-216.
114. Thornbury, W. D. (1954), "Principles of Geomorphology", John Wiley and Sons, N.Y. 618 pp.
115. Thornbury, W. D. (1965), "Regional Geomorphology of the United States", John Wiley and Sons, N.Y. 609 pp.
116. Transportation Research Board, (1976), Acquisition and Use of Geotechnical Information", Synthesis of Highway Practice 33, National Cooperative Highway Research Program, National Research Council, Washington, D.C. 40 pp.
117. Tschebotarioff, G. P. (1973), "Foundations, Retaining and Earth Structures - the Art of Design and Construction and its Scientific Basis in Soil Mechanics", 2nd Ed., McGraw-Hill Book Company, N.Y. pp. 365-414.
118. U.S. Navy (1971), "Design Manual: Soil Mechanics, Foundation, and Earth Structures", NAVFAC DM-7, Department of the Navy, Naval Facilities Engineering Command, Va., March. pp. 7-3-8, pp. 7-9-2, pp. 7-10-1 - 7-10-23.
119. Watanabi, S. and Imoto, M. (1962), "A Method of Determining the Consolidation Settlement from the Initial Void Ratio", Soil and Foundation, Japanese Society of Soil Mechanics and Foundation Engineering. 10 (9). pp. 7-11.
120. Weitzel, D. W. (1979), "The Effect of Laboratory Compaction on the Unconsolidated-Undrained Strength Behavior of a Highly Plastic Clay", MSCE Thesis, Purdue



University, West Lafayette, Indiana, August. See also Joint Highway Research Project Report No. 79-11, 218 pp.

121. Wiseman, G. and Zeitlen, J. G. (1961), "A Comparison between the CBR and the Shear Strength Methods in the Design of Flexible Pavements", Proc. 5th Int. Conf. on Soil Mech. and Foundation Engineering, Vol. II, Paris, pp. 359-363.
122. Woods, K. B. and Litehiser, R. R. (1938), "Soil Mechanics Applied to Highway Engineering in Ohio", Ohio State Univ. Eng. Expt. Station Bull. 99.
123. Wroth, C. P. and Wood, D. M. (1978), "The Correlation of Index Properties with Some Basic Engineering Properties of Soils", Canadian Geotechnical Journal, Vol. 15, No. 2, May, pp. 137-145.
124. Youssef, M. S., El Ramli, A. M., and El Demery, M. (1965), "Relationships between Shear Strength, Consolidation, Liquid Limit, and Plastic Limit for Remolded Clays", Proc. of the 6th Int. Conf. on Soil Mech. and Foundation Engineering, Montreal. P.Q., Vol. 1, pp. 126-129.



## APPENDICES

The following Appendices are not included in this copy of this Report:

<u>Appendix</u>	<u>Title</u>	<u>Pages</u>
A-I	Topographical Characteristics	155-162
A-II	Remolded Soil Characteristics	163-211
A-III	Statistical Soil Profiles	212-430
C	Multiple Comparison Tables	473- <sup>*</sup> 538

A copy of any of the above listed Appendices may be obtained for the cost of reproduction by writing to:

Joint Highway Research Project  
Civil Engineering Building  
Purdue University  
West Lafayette, Indiana 47907

---

\*Pages 459 through 472 of this Appendix are Codes referred to in this Report and are included in this copy of the Report for reference purposes.



APPENDIX A-IV  
REGRESSIONS EQUATIONS AND CORRELATIONS





A. Compaction parameters:

1. Maximum dry density ( $\rho_{d \max}$ ):

$$\begin{aligned}\hat{\rho}_{d \max} &= 135.843 - 1.279 w_p \text{ (in pcf)} \\ &= 2202.563 - 20.743 w_p \text{ (in kg/m}^3\text{)},\end{aligned}$$

for which  $R = -0.692$ , s.d. of est. = 6.434, and  $n = 601$ .

$$\begin{aligned}\hat{\rho}_{d \max} &= 128.338 - 0.463 w_L \text{ (in pcf)} \\ &= 2080.870 - 7.510 w_L \text{ (in kg/m}^3\text{)},\end{aligned}$$

for which  $R = 0.744$ , s.d. of est. = 5.651, and  $n = 601$ .

$$\begin{aligned}\hat{\rho}_{d \max} &= 142.888 - 0.554 w_L - 0.727 w_p + 0.00849 w_L w_p \\ &\quad - 0.0900 \text{ silt (in pcf)} \\ &= 2316.786 - 8.982 w_L - 11.787 w_p + 0.138 w_L w_p \\ &\quad - 1.459 \text{ silt (in kg/m}^3\text{)},\end{aligned}$$

for which  $|R| = 0.808$ , s.d. of est. = 4.994, and  $n = 601$ .

2. Optimum moisture content (OMC):

$$\hat{OMC} = 4.464 + 0.619 w_p \text{ (in \%)},$$

for which  $R = 0.698$ , s.d. of est. = 2.905, and  $n = 596$ .

$$\hat{OMC} = 7.626 + 0.237 w_L \text{ (in \%)},$$

for which  $R = 0.794$ , s.d. of est. = 2.464, and  $n = 596$ .

$$\begin{aligned}\hat{OMC} &= 7.457 = 0.0369 \text{ sand} + 0.0174 \text{ silt} + 0.171 w_L \\ &\quad + 0.155 w_p - 0.413 x_1 - 0.401 x_2 - 1.740 x_3 \\ &\quad + 0.968 x_4 - 0.409 x_5 + 0.503 x_6 - 0.186 x_7 \\ &\quad - 0.560 x_8 + 0.336 x_9 - 0.463 x_{10} - 0.752 x_{11} \\ &\text{(in \%)},\end{aligned}$$



for which  $|R| = 0.843$ , s.d. of est. = 2.210, and  $n = 596^*$ .

\*Note: No data available for physiographic region of Maumee Lacustrine Section ( $x_{12}$ ).

3. Maximum dry density vs. optimum moisture content:

$$\hat{\rho}_{d \max} = 150.667 - 3.016 \text{ OMC} + 0.0333 (\text{OMC})^2,$$

for which  $|R| = 0.906$ , s.d. of est. = 3.691,  $n = 701$ .

4. CBR at 100% maximum dry density (CBR S01):

$$\begin{aligned} \log \hat{\text{CBR}} \text{ S01} = & 1.204 + 0.145 \log w_L - 0.137 \log \text{PI} \\ & - 0.149 (\log \text{PI}) (\log w_L) - 0.0778 z_1 - 0.109 z_2 \\ & + 0.112 z_3 - 0.0505 z_4 - 0.0858 z_5 - 0.122 z_6 \\ & - 0.0575 z_7 - 0.0633 z_8 - 0.0587 z_9 - 0.105 z_{10} \\ & - 0.688 z_{11}, \end{aligned}$$

for which  $|R| = 0.620$ , s.d. of est. = 0.166, and  $n = 493$ .

5. CBR at 100% maximum dry density (CBR S01) vs. CBR at 95% maximum dry density (CBR S02):

$$\hat{\text{CBR}} \text{ S02} = 1.339 + 0.433 \text{ CBR S01},$$

for which  $R = 0.851$ , s.d. of est. = 2.010, and  $n = 553$ .

$$\hat{\text{CBR}} \text{ S02} = 0.051 + 0.667 \hat{\text{CBR}} \text{ S01} - 0.00760 (\text{CBR S01})^2,$$

for which  $|R| = 0.864$ , s.d. of est. = 1.515, and  $n = 553$ .

B. Consolidation parameters:

1. Compression index ( $C_c$ ):

$$\hat{C}_c = 0.00797 (w_L - 8.16),$$

for which  $R = 0.829$ , s.d. of est. = 0.116, and  $n = 312$ .

$$\hat{C}_c = 0.0126 w_n - 0.162,$$

for which  $R = 0.925$ , s.d. of est. = 0.112, and  $n = 332$ .



$$\hat{C}_c = 0.496 e_o - 0.195,$$

for which  $R = 0.873$ , s.d. of est. = 0.143, and  $n = 335$ .

$$\begin{aligned}\hat{C}_c = & -0.151 + 0.00326 w_n + 0.191 e_o + 0.00325 w_L \\ & + 0.0162 x_1 - 0.0110 x_2 + 0.0208 x_3 + 0.0296 x_4 \\ & + 0.0120 x_5 - 0.0110 x_6 + 0.0365 x_7 + 0.0351 x_8 \\ & + 0.0646 x_9 + 0.0649 x_{10} + 0.0594 x_{11} - 0.0245 z_1 \\ & - 0.0313 z_2 - 0.00987 z_3 - 0.0917 z_4 - 0.121 z_6 \\ & - 0.0292 z_7 - 0.0667 z_8 + 0.00841 z_9 - 0.0418 z_{10} \\ & - 0.00884 z_{11},\end{aligned}$$

for which  $|R| = 0.952$ , s.d. of est. = 0.0670, and  $n = 302^*$ .

\*Note: Note data available for physiographic region of Maumee Lacustrine Section ( $x_{12}$ ) and parent material of soils formed in loamy Wisconsin age glacial till ( $z_5$ ).

2. Compression ratio ( $C'_r$ ):

$$\hat{C}'_r = 0.0125 + 0.152 e_o,$$

for which  $R = 0.704$ , s.d. of est. = 0.0448, and  $n = 333$ .

$$\hat{C}'_r = 0.0249 + 0.003 w_n,$$

for which  $R = 0.701$ , s.d. of est. = 0.0361, and  $n = 325$ .

$$\hat{C}'_r = 0.0294 + 0.00238 w_L,$$

for which  $R = 0.665$ , s.d. of est. = 0.0373, and  $n = 309$ .

3. Compression index ( $C_c$ ) vs. compression ratio ( $C'_r$ ):

$$C_c = 0.0844 + 9.121 (C'_r)^2,$$

for which  $|R| = 0.948$ , s.d. of est. = 0.0928, and  $n = 339$ .

4. Recompression index ( $C_r$ ) vs. compression index ( $C_c$ ):

$$C_r = -0.00327 + 0.139 C_c,$$

for which  $R = 0.743$ , s.d. of est. = 0.0173, and  $n = 298$ .



### 5. Preconsolidation pressure ( $p_c$ ):

$$L1 = 0.770 - 0.115 \log p_c \text{ (Tsf)},$$

for which  $R = -0.200$ , s.d. of est. = 0.554, and  $n = 311$ .

$$\begin{aligned} \hat{p}_c &= 107.928 - 144.156 \log w_n + 30.687 (\log w_n)^2 \\ &\quad + 8.0064 (\log e_o)^2 - 3.448 (\log \rho_d)^3 \\ &\quad + 22.949 (\log w_n) (\log \rho_d) - 1.163 x_1 \\ &\quad - 1.336 x_2 + 0.240 x_3 - 0.932 x_4 - 1.297 x_5 \\ &\quad - 0.738 x_6 - 0.872 x_7 - 0.911 x_8 - 1.286 x_9 \\ &\quad + 0.195 x_{11} \text{ (} p_c \text{ in TsF and } \rho_d \text{ in pcf)} \\ &= 10340.00 - 13810.144 \log w_n + 2939.815 (\log w_n)^2 \\ &\quad + 767.013 (\log e_o)^2 - 330.318 (\log \frac{\rho_d}{16.214})^3 \\ &\quad + 2198.514 (\log w_n) (\log \frac{\rho_d}{16.214}) - 111.415 x_1 \\ &\quad - 127.989 x_2 + 22.992 x_3 - 89.286 x_4 - 124.253 x_5 \\ &\quad - 70.700 x_6 - 83.538 x_7 - 87.274 x_8 - 123.200 x_9 \\ &\quad + 18.681 x_{11} \text{ (} p_c \text{ in kPa and } \rho_d \text{ in kg/m}^3 \text{)}, \end{aligned}$$

for which  $|R| = 0.660$ , s.d. of est. = 705, and  $n = 243$ .\*

\*Note: No data available for physiographic regions of

Valparaiso Moraine and Maumee Lacustrine Section.

### C. Strength parameters:

#### 1. Unconfined compressive strength ( $q_u$ ):

$$\begin{aligned} \log \hat{q}_u &= 0.404 - 0.0205 w_n \text{ (in TsF)} \\ &= 2.385 - 0.0205 w_n \text{ (in kPa)}, \end{aligned}$$

for which  $R = -0.500$ , s.d. of est. = 0.254, and  $n = 1030$ .

$$\begin{aligned} \log \hat{q}_u &= -1.554 + 0.0144 \rho_d \text{ (} q_u \text{ in TsF and } \rho_d \text{ in pcf)} \\ &= 0.427 + 8.881 \times 10^{-4} \rho_d \text{ (} q_u \text{ in kPa and } \\ &\quad \rho_d \text{ in kg/m}^3 \text{)}, \end{aligned}$$

for which  $R = 0.522$ , s.d. of est. = 0.250, and  $n = 1030$ .





$$\log \hat{q}_u = -0.0425 - 0.264 \text{ LI (in Tsf)}$$

$$= 1.940 - 0.264 \text{ LI (in kPa)},$$

for which  $R = -0.366$ , s.d. of est. = 0.267, and  $n = 992$ .

$$\begin{aligned} \log \hat{q}_u &= -1.101 - 0.00740 w_n + 0.0112 \rho_d \\ &\quad + 0.0000319 w_n \rho_d - 0.0883 x_1 - 0.0518 x_2 \\ &\quad - 0.0224 x_3 - 0.0553 x_4 - 0.0568 x_5 \\ &\quad - 0.0440 x_6 - 0.0956 x_7 - 0.0346 x_8 \\ &\quad - 0.0140 x_9 + 0.0724 x_{11} + 0.172 z_1 \\ &\quad + 0.0568 z_2 + 0.109 z_3 + 0.0612 z_4 \\ &\quad - 0.163 z_5 - 0.165 z_6 + 0.0320 z_7 \\ &\quad + 0.173 z_8 - 0.0133 z_9 + 0.00343 z_{10} \\ &\quad + 0.0279 z_{11} (q_u \text{ in Tsf and } \rho_d \text{ in pcf}) \\ &= 0.880 - 0.00740 w_n + 6.910 \times 10^{-4} \rho_d \\ &\quad + 1.967 \times 10^{-6} w_n \rho_d - 0.0883 x_1 \\ &\quad - 0.0518 x_2 - 0.0224 x_3 - 0.553 x_4 \\ &\quad - 0.0568 x_5 - 0.0440 x_6 - 0.0956 x_7 \\ &\quad - 0.0346 x_8 - 0.0140 x_9 + 0.0724 x_{11} \\ &\quad + 0.0568 z_2 + 0.109 z_3 + 0.0612 z_4 \\ &\quad - 0.163 z_5 - 0.165 z_6 + 0.0320 z_7 \\ &\quad + 0.173 z_8 - 0.0133 z_9 + 0.00343 z_{10} \\ &\quad + 0.0279 z_{11} (q_u \text{ in kPa and } \rho_d \text{ in kg/m}^3), \end{aligned}$$

for which  $|R| = 0.570$ , s.d. of est. = 0.244, and  $n = 1030$ .\*

\*Note: No data available on physiographic regions of Valparaiso Moraine ( $x_{10}$ ) and Maumee Lacustrine Section ( $x_{12}$ ).



## 2. Liquid limit vs. SPT

$$\begin{aligned}
 w_L = & 47.946 + 0.495 w_n - 9.931 \log \text{SPT} \\
 & + 0.488 w_n \log \text{SPT} - 19.323 x_1 - 25.121 x_2 \\
 & - 30.455 x_3 - 17.775 x_4 - 28.875 x_5 \\
 & - 24.529 x_6 - 26.068 x_7 - 26.272 x_8 \\
 & - 24.100 x_9 - 16.493 x_{10} - 37.182 x_{11} \\
 & + 9.104 z_1 - 2.930 z_2 - 3.809 z_3 - 0.378 z_4 \\
 & - 12.178 z_5 - 5.404 z_6 - 7.021 z_7 + 23.069 z_8 \\
 & - 0.249 z_9 + 0.962 z_{10} + 13.878 z_{11} \text{ (in \%)},
 \end{aligned}$$

for which  $|R| = 0.859$ , s.d. of est. = 19.503,  $n = 533$ .

## 3. Unconfined compressive strength ( $q_u$ ) vs. SPT

$$\begin{aligned}
 q_u = & 0.577 + 0.0265 \text{ SPT (in Tsf)} \\
 = & 55.277 + 2.539 \text{ SPT (in kPa)},
 \end{aligned}$$

for which  $|R| = 0.400$ , s.d. of est. = 0.567, and  $n = 86$ .

## 4. Strength angle ( $\phi$ ) vs. SPT

$$\phi = 26.052 - 0.743 \text{ SPT (in degree)}$$

for which  $|R| = -0.450$ , s.d. of est. = 5.357, and  $n = 24$ .



APPENDIX B: CODING SYSTEMS AND  
COMPUTER PROGRAMS



## APPENDIX B-1

ERRATA TO THE USER'S MANUAL OF "THE  
DEVELOPMENT OF THE COMPUTERIZED GEO-  
TECHNICAL DATA BANK FOR THE STATE OF  
INDIANA" BY GARY D. GOLDBERG (35)





(a) The following coding system for soil association (ASSOC) is to replace the system as described in pp. 127-129, Goldberg (35).

---

<u>VARIABLE NAME</u>	<u>ASSOC</u>
<u>CODE</u>	<u>DESCRIPTION</u>
1.	Eel-Martinsville-Gemesee
2.	Genesee-Ross-Shoals
3.	Wakeland-Stendal-Haymond-Bartle
4.	Genesee-Shoals-Eel
5.	Haymond-Nolin-Petrolia
6.	Genesee-Eel-Stendal-Pope
7.	Huntington-Wheeling-Markland
8.	Huntington-Lindside
9.	Haymond-Wakeland
10.	Alida-Del Rey-Whitaker
11.	Bono-Maumee-Warners
12.	Chelsea-Hillsdale-Oshtemo
13.	Conrad-Wooten-Weiss
14.	Door-Tracy-Quinn
15.	Door-Lydick
16.	Elston-Wea
17.	Dubois-Otwell-Bartle
18.	Fox-Martinsville-Aluvi
19.	Fox-Nineveh-Ockley
20.	Fox-Rodman
21.	Fulton-Rimer-Milford-Rensselaer
22.	Homer-Sebawa-Gilford
23.	Maumee-Gilford-Rensselaer
24.	Maumee-Newton
26.	Martinsville-Bellmore-Fox
27.	Martinsville-Whitaker
28.	Mahalasville-Whitaker
30.	Milford-Montgomery-Rensselaer



<u>VARIABLE NAME</u>	ASSOC (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
31.	McGary
32.	Negley-Parke
33.	Oshtemo-Bronson
34.	Oakville-Plainfield-Adrian
35.	Oshtemo-Fox
36.	Ockley-Westland
37.	Ockley-Wea
38.	Ockley-Fox
39.	Plainfield-Brems-Morrocco
40.	Plainfield-Tyner-Oshtemo
41.	Plainfield- Watseka
42.	Plainfield-Chelsea
43.	Patton-Henshaw
44.	Patton-Lyles-Henshaw
45.	Peoga-Bartle-Hosmer
46.	Parke-Miami-Negley
47.	Rensselaer-Montgomery
48.	Rensselaer-Darroch
49.	Rensselaer-Whitaker
50.	Vincennes-Zipp-Ross
51.	Volinia
53.	Wea-Crane
54.	Warsaw-Elston-Fox
55.	Westland-Sleeth
56.	Weinbach-Sciotoville
57.	Weinbach-Wheeling
58.	Crosier-Brookston
59.	Brookston-Odell-Corwin
61.	Blount-Morley-Pewamo
62.	Blount-Pewamo
63.	Riddles-Miami-Crosier



<u>VARIABLE NAME</u>	ASSOC (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
64.	Crosby-Brookston
65.	Elliott-Markham-Pewamo
66.	Fincastle-Ragsdale-Brookston
67.	Hoytville-Nappanee
69.	Parr-Miami
70.	Parr-Corwin
71.	Randolph-Millsdale
72.	Reesville-Ragsdale
73.	Raub-Ragsdale
74.	Ragsdale-Sidell
76.	Russell-Hennepin
77.	Russell-Xenia
78.	Miami-Metea-Celina
79.	Miami-Owosso-Riddles
80.	Miami-Crosier-Metea
81.	Miami-Russell-Fincastle
82.	Miami-Fox-Milton
83.	Miami-Crosby
84.	Miami-Hennepin
85.	Miami-Fox-Martinsville
86.	Morley-Blount
87.	Muskingum-Shadeland-High Gap
88.	Odell-Chalmers
89.	Sidell-Parr
90.	Hennepin-Rodman
91.	Avonburg-Clermont
92.	Cincinnati-Hickory
93.	Cincinnati-Rossmoyne-Hickory
95.	Cincinnati-Ava
96.	Cincinnati-Ava-Alford
98.	Crider-Hagerstown-Frederick
99.	Crider-Frederick
100.	Corydon-Weikert-Berks



<u>VARIABLE NAME</u>	ASSOC (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
101.	Fairmount-Switzerland
102.	Grayford
103.	Lawrence-Bedford-Crider
104.	Tilsit-Johnsburg
105.	Wellston-Zanesville-Berks
106.	Berks-Gilpin-Weikert
107.	Zanesville-Wellston
108.	Mucks-Peats
109.	Alford
110.	Bloomfield-Princeton-Ayrshire
111.	Hosmer
112.	Iva-Ava
113.	Hosmer-Cincinnati-Iva
114.	Lyles-Ayrshire-Princeton
115.	Princeton-Ayrshire-Bloomfield
116.	Princeton-Fox





- (b) The following listing of computer program is to replace the Program Number 3 as described in pp. 152-160, Goldberg (35)

```

FILE NAME      PROG
RUN NAME      ***** SOIL DATA FOR THE STATE OF INDIANA *****
VARIABLE LIST  COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATMO,DATEDA,TOWN,TOWNDI,
               RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
               CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
               OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
               DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
               BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
               GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
               NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
               MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRSD1,CBRSD2,QUSTR,AASHTO1,UNIF1,
               QUSTA,TYPE,STRENGTH,STRAIN,CONFRES,COHESION,ANGLE,POREPRES,
               MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CU
INPUT MEDIUM  SOIL1
INPUT FORMAT  (1X,F2.0,FS.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
               1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  AS,A3,F3.0,1X,A3,
               FS.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/
               11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A8,A2, 1X,F2.0, 1X,F2.0, 1X, A8,
               1X,F5.1, 1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
               11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
               1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
               1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/ 11X,
               F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,
               1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
               11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
               1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
               1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
               F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
VAR LABELS    COUNTY COUNTY/HOLENO HOLE NUMBER/SAMPNO SAMPLE NUMBER/DATEYR YEAR
               TAKEN FROM HOLE/DATMO MONTH TAKEN FROM HOLE/DATEDA DAY TAKEN FR
               OM HOLE/TOWN TOWNSHIP/TOWNDI TOWNSHIP-DIRECTION/RANGE RANGE/
               RANGDI RANGE-DIRECTION/SECTIO SECTION/
               PROJPR PROJECT NUMBER-PREFIX/PROJNO PROJECT NUMBER/
               PROJPA PROJECT NUMBER-PARENTHESES/PROJMI PROJECT NUMBER-MILE/
               CONTPR CONTRACT NUMBER-PREFIX/
               CONTNO CONTRACT NUMBER/ DISTRI DISTRICT/
               ROADPR ROAD NUMBER-PREFIX/ROADNO ROAD NUMBER/
               ROADSU ROAD NUMBER-SUFFIX/BORING BORING NUMBER/ASSOC SOIL ASSOCIA
               TION/REPEAT DATA REPEAT/
               STATNO STATION NUMBER/OFFSET OFFSET/OFFDIR OFFSET-DIRECTION/
               LINE1 TO LINE2=LINE NUMBER/SOURCE SOURCE OF INFORMATION/
               SAMPTY SAMPLE TYPE/LABNO LAB NUMBER/
               GRDSUR GROUND SURFACE ELEVATION/DEPTHT DEPTH TO TOP OF SAMPLE/
               DEPTHB DEPTH TO BOTTOM OF SAMPLE/PHYSIO PHYSIOGRAPHIC UNIT/
               SPT N VALUE OF SPT/
               SERIES SOIL SERIES NAME/ PARENT PARENT MATERIAL/
               HORIZO HORIZON/ SLOPE SLOPE CLASS/ EROSION EROSION CLASS/
               BEDRKS DEPTH TO BEDROCK-SOIL SURVEY/ BEDRKB DEPTH TO BEDROCK-BO
               RING LOG/WATERS DEPTH-SEASONAL HIGH WATER TABLE-SOIL SURVEY/
               WATERC WATER DEPTH AT COMPLETION/WATERF WATER DEPTH FINAL OR 24 H
               OURS/DRAIN NATURAL SOIL DRAINAGE/PERMEA PERMEABILITY/
               FLOOD FLOODING POTENTIAL/ FROST POTENTIAL FROST ACTION/
               SHRINK SHRINK-SHELL POTENTIAL/ PH REACTION-PH/
               GRAD01 PERCENT PASSING 1 1-2# SIEVE/
               GRAD02 PERCENT PASSING 1# SIEVE/

```



GRAD03 PERCENT PASSING 3-4# SIEVE/  
 GRAD04 PERCENT PASSING 1-2# SIEVE/  
 GRAD05 PERCENT PASSING 3-8# SIEVE/  
 GRAD06 PERCENT PASSING NO. 4 SIEVE/  
 GRAD07 PERCENT PASSING NO. 10 SIEVE/  
 GRAD08 PERCENT PASSING NO. 40 SIEVE/  
 GRAD09 PERCENT PASSING NO. 200 SIEVE/  
 GRAD10 PERCENT PASSING NO. 270 SIEVE/  
 SAND PERCENT SAND/  
 SILT PERCENT SILT/CLAY PERCENT CLAY/COLL PERCENT COLLOIDS/  
 LL LIQUID LIMIT/ PL PLASTIC LIMIT/ PI PLASTICITY INDEX/  
 SL SHRINKAGE LIMIT/ LOSSIG LOSS ON IGNITION/  
 NATMC NATURAL MOISTURE CONTENT/ NATWD NATURAL WET DENSITY/  
 NATDD NATURAL DRY DENSITY/ SPECGR SPECIFIC GRAVITY/  
 TEXTUR TEXTURAL CLASSIFICATION/ ORGANI ORGANIC CONTENT/  
 COLOR COLOR/  
 TESTEF TEST-EFFORT IDENTIFIER/ MAXDD MAXIMUM DRY DENSITY/  
 MAXWD MAXIMUM WET DENSITY/ OPTIMC OPTIMUM MOISTURE CONTENT/  
 CBRUN1 UNSOAKED CBR-100 MAXDD/ CBRUN2 UNSOAKED CBR-95 MAXDD/  
 CBR501 SOAKED CBR-100 MAXDD/CBR502 SOAKED CBR-95 MAXDD/  
 QUSTR UNCONFINED COMPRESSIVE STRENGTH-TSF/  
 QUSTA FAILURE STRAIN-PERCENT/  
 TYPE TYPE OF STRENGTH TEST/STRENGTH FAILURE STRENGTH/  
 STRAIN FAILURE STRAIN/CONFRES CONFINING PRESSURE/  
 ANGLE FAILURE ANGLE/POREPRES PORE PRESSURE AT FAILURE/MAJOR MAJOR  
 PRINCIPAL STRESS/ED INITIAL VOID RATIO/EF FINAL VOID RATIO/SO IN  
 ITIAL DEGREE OF SATURATION/SF FINAL DEGREE OF SATURATION/PO OVERB  
 URDEN STRESS/PC PRECONSOLIDATION PRESSURE/CC COMPRESSION INDEX/  
 CR RECOMPRESSION INDEX/CU COEFFICIENT OF CONSOLIDATION  
 AASHTO1,UNIF1 (BLANK=9999)/ SAND (BLANK=999)  
 COUNTY (-0)UNKNOWN (01)ADAMS (02)ALLEN (03)BARTHOLOMEW (4)BENTON  
 (05)BLACKFORD (06)BOONE (07)BROWN (08)CARROLL (09)CASS (10)CLARK  
 (11)CLAY (12)CLINTON (13)CRAWFORD (14)DAVIESS (15)DEARBORN  
 (16)DECATUR (17)DEKALB (18)DELAWARE (19)DUBOIS (20)ELKHART  
 (21)FAYETTE (22)FLOYD (23)FOUNTAIN (24)FRANKLIN (25)FULTON  
 (26)GIBSON (27)GRANT (28)GREENE (29)HAMILTON (30)HANCOCK  
 (31)HARRISON (32)HENDRICKS (33)HENRY (34)HOWARD (35)HUNTINGTON  
 (36)JACKSON (37)JASPER (38)JAY (39)JEFFERSON (40)JENNINGS  
 (41)JOHNSON (42)KNOX (43)KOSCIUSKO (44)LAGRANGE (45)LAKE  
 (46)LAPORTE (47)LAWRENCE (48)MADISON (49)MARION (50)MARSHALL  
 (51)MARTIN (52)MIAMI (53)MONROE (54)MONTGOMERY (55)MORGAN  
 (56)NEWTON (57)NOBLE (58)OHIO (59)ORANGE (60)OWEN (61)PARKE  
 (62)PERRY (63)PIKE (64)PORTER (65)POSEY (66)PULASKI (67)PUTNAM  
 (68)RANDOLPH (69)RIPLEY (70)RUSH (71)ST. JOSEPH (72)SCOTT  
 (73)SHELBY (74)SPENCER (75)STARKE (76)STEUBEN (77)SULLIVAN  
 (78)SWITZERLAND (79)TIPPECANOE (80)TIPTON (81)UNION  
 (82)VANDERBURGH (83)VERMILLION (84)VIGO (85)WABASH (86)WARREN  
 (87)WARRICK (88)WASHINGTON (89)WAYNE (90)WELLS (91)WHITE  
 (92)WHITLEY (93)STATE OF KENTUCKY/  
 DISTRI (-0)UNKNOWN (1)CRAWFORDSVILLE  
 (2)FORT WAYNE (3)GREENFIELD (4)LAPORTE (5)SEYMOUR (6)VINCENNES/  
 DATEMO (-0)UNKNOWN (01)JANUARY (02)FEBRUARY (03)MARCH (04)APRIL  
 (05)MAY (06)JUNE (07)JULY (08)AUGUST (09)SEPTEMBER (10)OCTOBER  
 (11)NOVEMBER (12)DECEMBER/  
 TOWNDI (-0)UNKNOWN (1)NORTH (2)SOUTH/

RECODE  
 VALUE LABELS



RANGDI (-0)UNKNOWN (1)EAST (2)WEST/  
 ASSOC (-0)UNKNOWN  
 (1) EEL-MARTINSV-GENESSE (2) GENESEE-ROSS-SHOALS (3) WAKELAN-STEN  
 DA-HAYMO (4) GENESEE-SHOALS-EEL (5) HAYMON-NOLIN-PETROLI  
 (6) GENESE-EEL-STEND-POPE  
 (7) HUNTING-WHEELI-MARKH (8) HUNTINGTON-LINDSIDE  
 (9) HAYMOND-WAKELAND (10) ALIDA-DELREY-WHITAKE (11) BONO-MAUMEE  
 -WARNERS (12) CHELS-HILLSDAL-OSHT (13) CONRAD-WOOTEN-WEISS (14)  
 DOOR-TRACY-QUINN (15) DOOR-LYDICK (16) ELSTON-WEA (17) DUBOIS-OTW  
 ELL (18) FOX-MARTINSVILL-ALUV (19) FOX-NINEVEH-OCKLEY (20) FOX-RO  
 DMAN (21) FULT-RIM-MILF-RENSSEL (22) HOMER-SEBEWA-GILFORD (23) MA  
 UME-GILFORD-RENSSEL (24) MAUMEE-NEWTON (26) MARTINSVIL-BELMO-FOX  
 (27) MARTINSVILLE-WHITAKE (28) MAHALASVILL-WHITAKER (30) MILF-MONT  
 GOM-RENSSEL (31) MCGARY (32) NEGLEY-PARKE (33) OSHTMO-BRONSON  
 (34) DAKVIL-PLAINFIE-TAWA (35) OSHTMO-FOX (36) OCKLEY-WESTLAND  
 (37) OCKLEY-WEA (38) OCKLEY-FOX (39) PAILNFIE-BREM-MORROC (40) P  
 LAINFIE-TYNER-OSHT (41) PLAINFIELD-WATSEKA (42) PLAINFIELD-CHELS  
 EA (43) PATTON-HENSHAW (44) PATTON-LYLES-HENSHAW (45) PEOGA-BARTL  
 E-HOSMER (46) PARKE-MIAMI-NEGLEY (47) RENSSELAER-MONTGOMER (48)  
 RENSSELAER-DARROCH (49) RENSSELAER-WHITAKER (50) VINCENNES-ZIPP-R  
 OSS (51) VOLINIA-DICKINSON (53) WEA-CRANE (54) WARSAW-ELSTON-FOX  
 (55) WESTLAND-SLEETH (56) WEINBACH-SCIOTOVILLE (57) WEINBACH-WHEE  
 LING (58) CROSIER-BROOKSTON (59) BROOKSTO-ODELL-CORWI (61) BLOUNT  
 -MORLEY-PEWAMO (62) BLOUNT-PEWAMO (63) RIDDLES-MIAMI-CROSBY (64)  
 CROSBY-BROOKSTON (65) ELLIOT-MARKHAM-PEWAM (66) FINCASTLE-RAGSDAL  
 E (67) HOYTSVILLE-NAPPANEE (69) PARR-MIAMI (70) PARR-CORWIN (71)  
 RANDOLPH-HILLSDALE (72) REESVILLE-RAGSDALE (73) RAUB-RAGSDALE  
 (74) RAGSDALE-SIDELL (76) RUSSELL-HENNEPIN (77) RUSSELL-XENIA  
 (78) MIAMI-METEA-CELINA (79) MIAMI-OSOSSO-RIDDLES (80) MIAMI-C  
 ROSBY-METEA (81) MIA-RUSSEL-FINCASTLE (82) MIAMI-FOX-MILTON (83)  
 MIAMI-COSBY (84) MIAMI-HENNEPIN (85) MIAMI-FOX-MARTINSVIL (86) MO  
 RLEY-BLOUNT (87) MUSKIN-SHADELA-HIGHG (88) ODELL-CHALMERS (89) SI  
 DELL-PARR (90) HENNEPIN-RODMAN (91) AVONBURG-CLERMONT (92) CINCIN  
 NATI-HICKORY (93) CINCINNA-ROSSMO-HICK (95) CINCINNATI-AVA (96) C  
 INCINNAT-AVA-ALFORD (98) CRID-HAGERSTO-FREDER (99) CRIDER-FREDERI  
 CK (100) CORYDON-WEIKERT-BERK (101) FAIRMOUNT-SWITZERLAN (102) GR  
 AYFORD (103) LAWRENC-BEDFORD-CRID (104) TILSIT-JOHNSTOWN (105) WE  
 LLST-ZANESVIL-BERK (106) BERKS-GILPIN-WEIKERT (107) ZANESVILLE-WE  
 LLSTON (108) MUCKS-PEATS (109) ALFORD (110) BLOOMFI-PRINCE-AYRSH  
 (111) HOSMER (112) IVA-AVA (113) HOSMER-CINCINNAT-IVA (114) LYLE  
 -AYRSHIR-PRINCET (115) PRINCET-AYRSH-BLOOMF (116) PRINCETON-FOX/  
 REPEAT (1)ORIGINAL DATA (2)DATA FROM SAME HOLE  
 (3)DATA-DIFFERENT HOLE/  
 OFFDIR (-0)UNKNOWN (1)LEFT (2)RIGHT (3)CENTERLINE/  
 SOURCE (-0)UNKNOWN (01)STATE (02)ATEC-DEA  
 (03)ATEC-CONSULTANT (04)NUTTING-DEA (05)NUTTING-CONSULTANT  
 (06)PITTSBURG TEST-DEA (07)PITTSBURG TEST-CONS  
 (08)WESTENHOFF-NOV-DEA (09)WESTENHOFF-NOV-CONS (10)STOKLEY-DEA  
 (11)STOKLEY-CONSULTANT (12)STS-DEA (13)STS-CONSULTANT  
 (14)GEO SURVEYS-DEA (15)GEO SURVEYS-CONSULT. (16)TESTING SERVICE-  
 DEA (17)TESTING SERVICE-CONSULT. (18)HURST-ROSHE-DEA (19)HURST-RO  
 SHE-CONSULT. (20)CHASTAIN-DEA (21)CHASTAIN-CONSULTANT  
 (22)GREGG-DEA (23)GREGG-CONSULTANT (24)SHAFFER-DEA (25)SHAFFER-CONS  
 ULTANT/  
 SAMPTY (-0)UNKNOWN (01)SHELBY TUBE (02)SPLIT SPOON





(03)DENISON SAMPLER (04)CONT. FLIGHT AUGER (05)HAND AUGER  
 (06)JAR (07)BAG (08)ROCK CORE (09)PISTON SAMPLER (10)HOLLOW STEM  
 AUGER (11)POWER AUGER-MACHINE/  
 PHYSIO (-0)UNKNOWN (1)TIPTON TILL PLAIN (2)DEARBORN UPLAND  
 (3)MUSCATATUCK REGIONAL SLOPE (4)SCOTTSBURG LOWLAND  
 (5)NORMAN UPLAND (6)MITCHELL PLAIN (7)CRAWFORD UPLAND  
 (8)WABASH LOWLAND (9)CALUMET LACUSTRINE (10)VALPARAISO MORAINE  
 (11)KANKAKEE LACUSTRINE (12)MAUMEE LACUSTRINE (13)STEUBEN MORAINA  
 L/  
 SERIES (-0)UNKNOWN (10)ADE (20)ADRIAN (30)ALFORD (40)ALGIERS  
 (50)ALIDA (60)ALLISON (70)ARMIESBURG (80)AUBBEENAUBBEE  
 (90)AVA (100)AVONBURG (110)AYR (120)AYRSHERE (130)BARTLE  
 (140)BAXTER (150)BEDFORD (160)BELLMORE (170)BERKS  
 (180)BIRDS (190)BLOOMFIELD (200)BLOUNT (210)BONNIE  
 (220)BOND (230)BOONESBORO (240)BOYER (250)BRADY  
 (260)BREMS (270)BRONSON (280)BROOKSTON (290)BURGIN  
 (300)BURNSIDE (310)CAMDEN (320)CARLISLE (330)CASCO  
 (340)CATLIN (350)CELINA (360)CHALMERS (370)CHELSEA  
 (380)CINCINNATI (390)CLARENCE (400)CLERMONT (410)COLYER  
 (420)CONOVER (430)CONRAD (440)CORWIN (450)CORY (460)CORYD  
 ON (470)COUPEE (480)CRANE (490)CRIDER (500)CROSBY  
 (510)CROSIER (520)CUBA (530)DANA (540)DARROCH (550)DEL RAY  
 (560)DICKINSON (570)DOOR (580)DOWAGIAC (590)DUBOIS (600)D  
 UNNING (610)EDEN (620)EDENTON (630)EDWARDS (640)EEL  
 (650)ELKINSONVILLE (660)ELLIOTT (670)ELSTON (680)EVANSVILLE  
 (690)FABIUS (700)FAIRMOUNT (710)FINCASTLE (720)FLANAGAN  
 (730)FORESMAN (740)FOX-SILT LOAM (741)FOX-LOAM (742)FOX-URBAN LAN  
 D (750)FREDERICK (760)FULTON (770)GENESEE-SILT LOAM (771)GENESEE-  
 URBAN LAND (780)GILFORD (790)GILPIN (800)GINAT (810)GLENHALL  
 (820)GRANBY (830)GRAYFORD (840)GUTHRIE (850)HAGERSTOWN  
 (860)HANNA (870)HASKINS (880)HAUBSTADT (890)HAYMOND (900)  
 HENNEPIN (910)HENSHAW (920)HICKORY (930)HIGH GAP (940)HIL  
 LSDALE (950)HOMER (960)HOOPESTON (970)HOSMER (980)HOUGHTO  
 N (990)HOYTVILLE (1000)HUNTINGTON (1010)HUNTSVILLE (1020)  
 IONA (1030)IPAUA (1040)IVA (1050)JASPER (1060)JENNINGS  
 (1070)JOHNSBURG (1080)JULES (1090)KALAMAZOO (1100)KERSTON  
 (1110)KINGS (1120)KOKOMO (1130)LANDES (1140)LAWRENCE  
 (1150)LENAWEE (1160)LINDSIDE (1170)LINKVILLE (1180)LONGLOIS  
 (1190)LORENZO (1200)LOWELL (1210)LUCAS (1220)LYDICK (1230)  
 LYLES (1240)MAHALASVILLE (1250)MARKHAM (1260)MARKLAND  
 (1270)MARTINSVILLE (1280)MARTISCO (1290)MASSIE (1300)MATHER  
 TON (1310)MAUMEE (1320)MCGARY (1330)MEDWAY (1340)MELLOTT  
 (1350)MERMILL (1360)METAMORA (1370)METEA (1380)MIAMI-SILT L  
 OAM (1381)MIAMI-URBAN LAND  
 (1390)MILFORD (1400)MILLSDALE (1410)MILTON (1420)MONITOR  
 (1430)MONTGOMERY (1440)MONTMORENCI (1450)MORLEY (1460)MOROC  
 CO (1470)MUREN (1480)MUSKINGUM (1490)MUSSEY (1500)NAPPANE  
 E (1510)NEGLEY (1520)NEWARK (1530)NEWTON (1540)NICHOLSON  
 (1550)NINEVEH (1560)NOLIN (1570)OAKVILLE (1580)OCKLEY  
 (1590)OCTAGON (1600)ODELL (1610)OSHTIMO (1620)OTWELL  
 (1630)OWOSSO (1640)PALMS (1650)PARKE (1660)PARR (1670)PATT  
 ON (1680)PEKIN (1690)PEOGA (1700)PETROLIA (1710)PEWAMO  
 (1720)PIKE (1730)PINHOOK (1740)PLAINFIELD (1750)PLANO  
 (1760)POPE (1770)PRINCETON (1780)PROCTOR (1790)QUINN  
 (1800)RAGSDALE (1810)RAHM (1820)RANDOLPH (1830)RARDEN





(1840)RAUB (1850)RAWSON (1860)REESVILLE (1870)RENSSELAER  
 (1880)RIDDLES (1890)RIMER (1900)ROBINSON (1910)ROCKCASTLE  
 (1920)RODMAN (1930)ROSS (1940)ROSSMOYNE (1950)RUNNYMEDE  
 (1960)RUSH (1970)RUSSELL (1980)ST. CLAIR (1990)SARANAC  
 (2000)SAUGATUCK (2010)SCIOTOVILLE (2020)SEBEWA (2030)SEWARD  
 (2040)SHADELAND (2050)SHIPSHE (2060)SHOALS (2070)SIDELL  
 (2080)SLEETH (2090)SLOAN (2100)SPARTA (2110)STARKS  
 (2120)STEFF (2130)STENDAL (2140)STONELICK (2150)STOY  
 (2160)STROLE (2170)SUNBURY (2180)SWITZERLAND (2190)SWYGER  
 (2200)SYLVAN (2210)TAGGART (2220)TAMA (2230)TEDROW (2240)  
 TILSIT (2250)TIPPECANOE (2260)TOLEDO (2270)TORONTO (2280)  
 TRACY (2290)TRAPPIST (2300)TROXEL (2310)TYNER (2320)UNION  
 TOWN (2330)UIGO (2340)VINCENNES (2350)VOLINIA (2360)WAKEL  
 AND (2370)WALLKILL (2380)WARNERS (2390)WARSAW (2400)WASEP  
 I (2410)WASHTENAW (2420)WATSEKA (2430)WAUSEON (2440)WEA  
 (2450)WEIKERT (2460)WEINBACH (2470)WELLSTON (2480)WESTLAND  
 (2490)WHEELING (2500)WHITAKER (2510)WHITSON (2520)WILBUR  
 (2530)WILLETTE (2540)WINGATE (2550)WOODMERE (2560)WOOLPER  
 (2570)WYNN (2580)XENIA (2590)ZANESVILLE (2600)ZIPP  
 (2700)BORROW PIT (2800)URBAN LAND (2900)ALLUVIAL LAND  
 (3000)GRAVEL PIT (3100)GULLIED LAND (3200)STRIP MINE  
 (3300)CUT AND FILL (3400)MADE LAND/  
 PARENT (-0)UNKNOWN (1)LACUSTRINE DEPOSITS (2)OUTWASH AND ALLUVIAL  
 DEPOSITS (3)EOLIAN SAND DEPOSITS (4)THICK LOESS DEPOSITS (5)LOAMY  
 WISCONSIN TILL (6)CLAYEY WIS. TILL (7)THIN LOESS-LOAMY WIS TILL  
 (8)THICK LOESS-LOAM WIS TILL (9)THICK LOESS-ILL TILL  
 (10)RESIDUUM-SI. ST SH SS (11)RESIDUUM-LS BEDROCK  
 (12)RES-CALCAREOUS SH LS/  
 HORIZO (-0)UNKNOWN (1)A HORIZON (2)B HORIZON (3)C HORIZON  
 (4)D HORIZON/  
 SLOPE (-0)UNKNOWN (1)LEVEL-NEARLY LEVEL (2)0-2 (3)2-6 (4)6-12  
 (5)12-18 (6)18-25 (7)25-35 (8)35+/  
 EROSID (-0)UNKNOWN (1)NONE-SLIGHT (2)MODERATE-ERODED (3)SEVERE/  
 WATERC (-0)UNKNOWN (-1)DRY/  
 WATERF (-0)UNKNOWN (-1)DRY/  
 DRAIN (-0)UNKNOWN (1)WELL-EXCESSIVE (2)MODERATELY WELL  
 (3)SOMEWHAT POORLY (4)POORLY-VERY POORLY/  
 PERMEA (-0)UNKNOWN (1)LESS THAN .06 (2).06-.2 (3).2-.63 (4).63-2.  
 0 (5)2.0-6.0 (6)6.0-20 (7)GREATER THAN 20/  
 FLOOD (-0)UNKNOWN (1)NONE (2)PERCHED-PONDED-HAZARD/  
 FROST (-0)UNKNOWN (1)VERY LOW (2)VERY LOW-LOW (3)LOW (4)LOW-MODE  
 RATE (5)MODERATE (6)MODERATE-HIGH (7)HIGH (8)HIGH-VERY HIGH  
 (9)VERY HIGH/  
 SHRINK (-0)UNKNOWN (1)VERY LOW (2)VERY LOW-LOW (3)LOW (4)LOW-MODE  
 RATE (5)MODERATE (6)MODERATE-HIGH (7)HIGH (8)HIGH-VERY HIGH  
 (9)VERY HIGH/  
 PH (-0)UNKNOWN (01)BELOW 4.5 (02)4.5-5.0 (03)5.1-5.5 (04)5.6-6.  
 0 (05)6.1-6.5 (06)6.6-7.3 (07)7.4-7.8 (08)7.9-8.4 (09)8.5-9.0  
 (10)ABOVE 9.0/  
 LL, PL, PI, SL (-0)UNKNOWN (-1)NOT PLASTIC/  
 TEXTUR (-0)UNKNOWN (1)SAND (2)SAND-TRACE GRAVEL (3)SAND-LITTLE GR  
 AVEL (4)SAND-SOME GRAVEL (5)SAND AND GRAVEL (6)SANDY LOAM  
 (7)SANDY LOAM-TRACE GRAVEL (8)SANDY LOAM-LITTLE GRAVEL  
 (9)SANDY LOAM-SOME GRAVEL (10)SANDY LOAM AND GRAVEL  
 (11)LOAM (12)LOAM-TRACE GRAVEL (13)LOAM-LITTLE GRAVEL



(14)LOAM-SOME GRAVEL (15)LOAM AND GRAVEL (16)SILTY LOAM  
 (17)SILTY LOAM-TRACE GRAVEL (18)SILTY LOAM-LITTLE GRAVEL  
 (19)SILTY LOAM-SOME GRAVEL (20)SILTY LOAM AND GRAVEL  
 (21)SILT (22)SILT-TRACE GRAVEL (23)SILT-LITTLE GRAVEL  
 (24)SILT-SOME GRAVEL (25)SILT AND GRAVEL (26)SANDY CLAY LOAM  
 (27)SANDY CLAY LOAM-TRACE GRAVEL (28)SANDY CLAY LOAM-LITTLE GRAVE  
 L (29)SANDY CLAY LOAM-SOME GRAVEL (30)SANDY CLAY LOAM AND GRAVEL  
 (31)CLAY LOAM (32)CLAY LOAM-TRACE GRAVEL (33)CLAY LOAM-LITTLE GRA  
 VEL (34)CLAY LOAM-SOME GRAVEL (35)CLAY LOAM AND GRAVEL  
 (36)SILTY CLAY LOAM (37)SILTY CLAY LOAM-TRACE GRAVEL  
 (38)SILTY CLAY LOAM-LITTLE GRAVEL (39)SILTY CLAY LOAM-SOME GRAVEL  
 (40)SILTY CLAY LOAM AND GRAVEL (41)SANDY CLAY  
 (42)SANDY CLAY-TRACE GRAVEL (43)SANDY CLAY-LITTLE GRAVEL  
 (44)SANDY CLAY-SOME GRAVEL (45)SANDY CLAY AND GRAVEL  
 (46)SILTY CLAY (47)SILTY CLAY-TRACE GRAVEL (48)SILTY CLAY-LITTLE  
 GRAVEL (49)SILTY CLAY-SOME GRAVEL (50)SILTY CLAY AND GRAVEL  
 (51)CLAY (52)CLAY-TRACE GRAVEL (53)CLAY-LITTLE GRAVEL  
 (54)CLAY-SOME GRAVEL (55)CLAY AND GRAVEL  
 (56)GRAVEL (57)SANDY GRAVEL (58)GRAVELLY SAND  
 (59)SAND AND GRAVEL  
 (60)COPROGENOUS EARTH (61)DIATOMACEOUS EARTH (62)FIBRIC MATERIAL  
 (63)FRAGMENTAL MATERIAL (64)HEMIC MATERIAL (65)ICE OR FROZEN SOIL  
 (66)MARL (67)MUCK (68)MUCKY PEAT (69)OXIDE-PROTECT. WX BR  
 (70)PART. DECOM ORG MATL (71)PEAT (72)SAPRIC MATERIAL  
 (73)UNDECOM ORG MATL (74)UNWEATHERED BEDROCK  
 (75)WX BR,SAPROLITE,GRUS (76)COMPLEX (77)INAPPLICABLE/  
 ORGANI (-0)UNKNOWN (1)NO ORGANIC MATL (2)TRACE (3)LITTLE (4)SOME  
 (5)AND/  
 COLOR (-0)UNKNOWN (01)BLUISH GRAY (02)BLACK  
 (03)BROWN (04)BROWNISH YELLOW (05)DARK BROWN (06)DARK BLUISH GRAY  
 (07)DARK GRAY (08)DARK GRAYISH BROWN (09)DARK GREENISH GRAY  
 (10)DARK OLIVE (11)DARK OLIVE GRAY (12)DARK RED  
 (13)DARK REDDISH BROWN (14)DARK REDDISH GRAY (15)DUSKY RED  
 (16)DARK YELLOWISH BROWN (17)GRAYISH BROWN (18)GREENISH GRAY  
 (19)GRAYISH GREEN (20)GREEN (21)GRAY (22)LIGHT BLUISH GRAY  
 (23)LIGHT BROWN (24)LIGHT BROWNISH GRAY (25)LIGHT GREENISH GRAY  
 (26)LIGHT GRAY (27)LIGHT OLIVE BROWN (28)LIGHT OLIVE GRAY  
 (29)LIGHT RED (30)LIGHT REDDISH BROWN (31)LIGHT YELLOWISH BR.  
 (32)OLIVE BROWN (33)OLIVE (34)OLIVE GRAY (35)OLIVE YELLOW  
 (36)PALE BROWN (37)PALE GREEN (38)PALE OLIVE (39)PALE RED  
 (40)PALE YELLOW (41)PINKISH GRAY (42)PINK (43)PINKISH WHITE  
 (44)REDDISH BLACK (45)REDDISH BROWN (46)RED (47)REDDISH GRAY  
 (48)REDDISH YELLOW (49)STRONG BROWN (50)VERY DARK BROWN  
 (51)VERY DARK GRAY (52)U. DK. GRAYISH BROWN (53)VERY DARK RED  
 (54)VERY PALE BROWN (55)VERY DUSKY RED (56)WEAK RED (57)WHITE  
 (58)YELLOWISH BROWN (59)YELLOWISH RED (60)YELLOW/  
 TESTEF (-0)UNKNOWN (01)STANDARD-12400  
 (02)STANDARD-SEE DIF (03)STANDARD-NOT GIVEN (04)MODIFIED-12400  
 (05)MODIFIED-56000 (06)MODIFIED-56300 (07)MODIFIED-SEE DIF  
 (08)MODIFIED-NOT GIVEN (09)15 BLOW-7400 (10)15 BLOW-7800  
 (11)15 BLOW-SEE DIF (12)15 BLOW-NOT GIVEN (13)KNEADING-SEE DIF  
 (14)KNEADING-NOT GIVEN (15)HARVARD MIN-SEE DIF  
 (16)HARVARD MIN-NOT GIVEN (17)HUEEM-SEE DIF (18)HUEEM-NOT GIVEN  
 (19)VIBRATORY-SEE DIF (20)VIBRATORY-NOT GIVEN (21)COMPRESSION-SEE  
 DIF (22)COMPRESSION-NOT GIVEN/



TYPE (-0)UNKNOWN (1)UU TEST (2)CU TEST-UNSATURATED  
 (3)CU TEST-SATURATED (4)DIRECT SHEAR (5)CD TEST/  
 HOLENO,SAMPNO,DATEYR,DATEDA,TOWN,RANGE,SECTID TO BORING,STATNO,  
 OFFSET,LINE1,LINE2,LABNO TO SPT,BEDRKS TO WATERS,GRAD01 TO COLL,  
 LOSSIG TO SPECGR,MAXDD TO QUSTR,QUSTA,STRENGTH TO CU  
 (-0)UNKNOWN

MISSING VALUES	COUNTY TO QUSTR,QUSTA TO CU (-0)/
COMMENT	THE FOLLOWING SERIES OF ≠IF≠ STATEMENTS ARE TO GIVE THE CODED DESIGNATIONS OF ORGANIC CONTENTS
IF	(LOSSIG GT 0.001 AND LE 1) ORGANI=1
IF	(LOSSIG GT 1 AND LE 10) ORGANI=2
IF	(LOSSIG GT 10 AND LE 20) ORGANI=3
IF	(LOSSIG GT 20 AND LE 35) ORGANI=4
IF	(LOSSIG GT 35) ORGANI=5
COMMENT	THE FOLLOWING SERIES OF ≠IF≠ STATEMENTS ARE TO CORRECT THE PHYSIOGRAPHIC REGION PERTAINING TO A COUNTY
IF	(COUNTY EQ 65) PHYSIO=8
IF	(COUNTY EQ 27) PHYSIO=8
IF	(COUNTY EQ 79) PHYSIO=1
COMMENT	THE FOLLOWING SERIES OF ≠IF≠ STATEMENTS ARE TO GIVE CORRECT PARENT MATERIALS PERTAINING TO SOIL ASSOCIATIONS
IF	(ASSOC EQ 55) PARENT=2
IF	(ASSOC EQ 56) PARENT=2
IF	(ASSOC EQ 57) PARENT=2
IF	(ASSOC EQ 58) PARENT=5
IF	(ASSOC EQ 59) PARENT=7
IF	(ASSOC EQ 61) PARENT=6
IF	(ASSOC EQ 62) PARENT=6
IF	(ASSOC EQ 63) PARENT=5
IF	(ASSOC EQ 64) PARENT=7
IF	(ASSOC EQ 65) PARENT=6
IF	(ASSOC EQ 66) PARENT=8
IF	(ASSOC EQ 67) PARENT=6
IF	(ASSOC EQ 69) PARENT=7
IF	(ASSOC EQ 70) PARENT=7
IF	(ASSOC EQ 71) PARENT=7
IF	(ASSOC EQ 72) PARENT=8
IF	(ASSOC EQ 73) PARENT=8
IF	(ASSOC EQ 74) PARENT=8
IF	(ASSOC EQ 76) PARENT=8
IF	(ASSOC EQ 77) PARENT=8
IF	(ASSOC EQ 78) PARENT=7
IF	(ASSOC EQ 79) PARENT=5
IF	(ASSOC EQ 80) PARENT=5
IF	(ASSOC EQ 81) PARENT=8
IF	(ASSOC EQ 82) PARENT=7
IF	(ASSOC EQ 83) PARENT=7
IF	(ASSOC EQ 84) PARENT=7
IF	(ASSOC EQ 85) PARENT=2
IF	(ASSOC EQ 86) PARENT=6
IF	(ASSOC EQ 87) PARENT=7
IF	(ASSOC EQ 88) PARENT=7
IF	(ASSOC EQ 89) PARENT=7
IF	(ASSOC EQ 90) PARENT=7
IF	(ASSOC EQ 91) PARENT=9





```

IF      (ASSOC EQ 92) PARENT=9
IF      (ASSOC EQ 93) PARENT=9
IF      (ASSOC EQ 95) PARENT=9
IF      (ASSOC EQ 96) PARENT=9
IF      (ASSOC EQ 98) PARENT=11
IF      (ASSOC EQ 99) PARENT=11
IF      (ASSOC EQ 100) PARENT=10
IF      (ASSOC EQ 101) PARENT=12
IF      (ASSOC EQ 102) PARENT=9
IF      (ASSOC EQ 103) PARENT=11
IF      (ASSOC EQ 104) PARENT=10
IF      (ASSOC EQ 105) PARENT=10
IF      (ASSOC EQ 106) PARENT=10
IF      (ASSOC EQ 107) PARENT=10
IF      (ASSOC EQ 108) PARENT=1
IF      (ASSOC EQ 109) PARENT=4
IF      (ASSOC EQ 110) PARENT=3
IF      (ASSOC EQ 111) PARENT=4
IF      (ASSOC EQ 112) PARENT=4
IF      (ASSOC EQ 113) PARENT=4
IF      (ASSOC EQ 114) PARENT=3
IF      (ASSOC EQ 115) PARENT=3
IF      (ASSOC EQ 116) PARENT=3
COMMENT THE FOLLOWING SERIES OF ~IF~ STATEMENTS COMPUTE BOTH
        THE AASHTO AND UNIFIED CLASSIFICATION CODES-FOR THOSE
        PROGRAMS NOT REQUIRING EITHER OF THESE TWO CLASSIFICATIONS,
        THESE STATEMENTS SHOULD BE OMITTED
IF      (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) AASHTO=13
IF      (GRAD09 GE 35.5 AND LL GE 40.5 AND PI GE 10.5 AND PI GT (LL-30))
        AASHTO=12
IF      (GRAD09 GE 35.5 AND LL GE 40.5 AND PI GE 10.5 AND PI LE (LL-30))
        AASHTO=11
IF      (GRAD09 GE 35.5 AND LL LT 40.5 AND PI GE 10.5) AASHTO=10
IF      (GRAD09 GE 35.5 AND (LL GE 40.5 OR EQ -1) AND PI LT 10.5)
        AASHTO=09
IF      (GRAD09 GE 35.5 AND LL LT 40.5 AND PI LT 10.5) AASHTO=08
IF      (GRAD09 LT 35.5 AND LL GE 40.5 AND PI GE 10.5) AASHTO=06
IF      (GRAD09 LT 35.5 AND LL LT 40.5 AND PI GE 10.5) AASHTO=05
IF      (GRAD09 LT 35.5 AND (LL GE 40.5 OR EQ -1) AND PI LT 10.5)
        AASHTO=04
IF      (GRAD09 LT 35.5 AND LL LT 40.5 AND PI LT 10.5) AASHTO=03
IF      (GRAD08 GE 50.5 AND GRAD09 LT 10.5 AND (LL EQ -1 OR PL EQ -1
        OR PI EQ -1)) AASHTO=07
IF      (GRAD08 LT 50.5 AND GRAD09 LT 25.5 AND PI LT 6.5) AASHTO=02
IF      (GRAD07 LT 50.5 AND GRAD08 LT 30.5 AND GRAD09 LT 15.5
        AND PI LT 6.5) AASHTO=01
IF      (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) AASHTO=13
IF      (AASHTO NE 05 OR 06)
        GI=RND((GRAD09-35)*(.2+.005*(LL-40))+.01*(GRAD09-15)*(PI-10))
IF      (AASHTO EQ 05 OR 06) GI=RND(.01*(GRAD09-15)*(PI-10))
IF      (PL EQ -1 OR LL EQ -1 OR GI LT 0) GI=0
ASSIGN MISSING GI (-8)
IF      (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) UNIF=35
IF      (GRAD09 GE 50 AND (PI GT (.73*(LL-20)) AND 7) AND LL LT 50)
        UNIF=01

```





```

IF      (GRAD09 GE 50 AND PI GT (.73*(LL-20)) AND LL GT 50) UNIF=02
IF      (GRAD09 GE 50 AND PI GT (.73*(LL-20)) AND LL EQ 50) UNIF=03
IF      (GRAD09 GE 50 AND (PI LT (.73*(LL-20)) OR 4) AND LL LT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=04
IF      (GRAD09 GE 50 AND (PI LT (.73*(LL-20)) OR 4) AND LL LT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=05
IF      (GRAD09 GE 50 AND (PI LT (.73*(LL-20)) OR 4) AND LL LT 50 AND
        ORGANI EQ 3) UNIF=06
IF      (GRAD09 GE 50 AND (PI EQ (.73*(LL-20)) AND GT 7) AND LL LT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=07
IF      (GRAD09 GE 50 AND (PI EQ (.73*(LL-20)) AND GT 7) AND LL LT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=08
IF      (GRAD09 GE 50 AND (PI EQ (.73*(LL-20)) AND GT 7) AND LL LT 50 AND
        ORGANI EQ 3) UNIF=09
IF      (GRAD09 GE 50 AND PI LT (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=10
IF      (GRAD09 GE 50 AND PI LT (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=11
IF      (GRAD09 GE 50 AND PI LT (.73*(LL-20)) AND LL GT 50 AND ORGANI EQ
        3) UNIF=12
IF      (GRAD09 GE 50 AND PI EQ (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=13
IF      (GRAD09 GE 50 AND PI EQ (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=14
IF      (GRAD09 GE 50 AND PI EQ (.73*(LL-20)) AND LL GT 50 AND ORGANI EQ
        3) UNIF=15
IF      (GRAD09 GE 50 AND PI LT (.73*(LL-20)) AND LL EQ 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=16
IF      (GRAD09 GE 50 AND PI LT (.73*(LL-20)) AND LL EQ 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=17
IF      (GRAD09 GE 50 AND PI LT (.73*(LL-20)) AND LL EQ 50 AND ORGANI EQ
        3) UNIF=18
IF      (GRAD09 GE 50 AND PI EQ (.73*(LL-20)) AND LL EQ 50) UNIF=19
IF      (GRAD09 GE 50 AND (PI GE (.73*(LL-20)) AND LE 7 AND GE 4))
        UNIF=20
IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND GRAD09 GT 12 AND (PI LT
        (.73*(LL-20)) OR 4)) UNIF=21
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND GRAD09 GT 12 AND (PI LT
        (.73*(LL-20)) OR 4)) UNIF=24
IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND GRAD09 GT 12 AND (PI GT
        (.73*(LL-20)) AND 7)) UNIF=22
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND GRAD09 GT 12 AND (PI GT
        (.73*(LL-20)) AND 7)) UNIF=25
IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND GRAD09 GT 12 AND ((PI EQ
        (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND GE 4)))
        UNIF=23
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND GRAD09 GT 12 AND ((PI EQ
        (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND GE 4)))
        UNIF=26
IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND GRAD09 LT 5) UNIF=27
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND GRAD09 LT 5) UNIF=28
IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND (GRAD09 GE 5 AND LE 12) AND
        (PI LT (.73*(LL-20)) OR 4)) UNIF=29
IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND (GRAD09 GE 5 AND LE 12) AND
        (PI GT (.73*(LL-20)) AND 7)) UNIF=30

```



```

IF      (GRAD09 LT 50 AND GRAD06 LE 50 AND (GRAD09 GE 5 AND LE 12) AND
        ((PI EQ (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND
        GE 4))) UNIF=31
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND (GRAD09 GE 5 AND LE 12) AND
        (PI LT (.73*(LL-20)) OR 4)) UNIF=32
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND (GRAD09 GE 5 AND LE 12) AND
        (PI GT (.73*(LL-20)) AND 7)) UNIF=33
IF      (GRAD09 LT 50 AND GRAD06 GT 50 AND (GRAD09 GE 5 AND LE 12) AND
        ((PI EQ (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND
        GE 4))) UNIF=34
IF      (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) UNIF=35
IF      (AASHTO1 NE 9999) AASHTO=AASHTO1
IF      (UNIF1 NE 9999) UNIF=UNIF1
IF      (SAND EQ 999) SAND=-0
IF      (SAND NE 999) SAND=GRAD07-GRAD09
ASSIGN MISSING AASHTO,UNIF (-8)
VAR LABELS    GI GROUP INDEX/AASHTO AASHTO CLASSIFICATION/UNIF UNIFIED CLASSIFI
               CATION
VALUE LABELS  GI (-8)MISSING DATA/
               AASHTO (-0)UNKNOWN (1)A-1-A (2)A-1-B (3)A-2-4 (4)A-2-5 (5)A-2-6
               (6)A-2-7 (7)A-3 (8)A-4 (9)A-5 (10)A-6 (11)A-7-5 (12)A-7-6
               (13)A-8/
               UNIF (-0)UNKNOWN (01)CL (02)CH (03)CL-CH (04)ML (05)OL (06)ML-OL
               (07)ML-CL (08)CL-OL (09)ML-OL OR CL (10)MH (11)OH (12)MH-OH
               (13)MH-CH (14)CH-OH (15)MH-OH OR CH (16)ML-MH (17)OL-OH
               (18)MH-OH OR ML-OL (19)ANY COMBINATION (20)CL-ML
               (21)GM (22)GC (23)GM-GC (24)SM (25)SC (26)SM-SC (27)GW OR GP*=G
               (28)SW OR SP*=S (29)G-GM (30)G-GC (31)G-GM OR G-GC (32)S-SM
               (33)S-SC (34)S-SM OR S-SC (35)PT
COMMENT       THE ABOVE STATEMENT IS THE LAST STATEMENT WHICH
               SHOULD BE OMITTED IF THE AASHTO AND UNIFIED
COMMENT       CLASSIFICATIONS ARE NOT REQUIRED
               THE NUMBER OF CASES (N OF CASES) WILL COINCIDE WITH
               THE NUMBER OF SAMPLES CONTAINED WITHIN THE DATA BANK
               AT THE TIME OF PROCESSING
N OF CASES    9442
PRINT FORMATS SPECGR,ED,EF,CC,CR (3)/ QUSTR,QUSTA,STRENGTH TO COHESION,
               POREPRESS,MAJOR,PO,PC,CU (2)/ GRDSUR TO DEPTH,
               BEDRKS TO WATERF,
               GRAD01 TO NATDD,MAXDD TO CBR502,ANGLE,SO,SF (1)/PROJPR,
               PROJPA,CONTPR,ROADPR,ROADSU TO BORING,LINE1,LINE2,LABNO (A)
READ INPUT DATA
SAVE FILE     PROG
FINISH
6/7/8/9

```



APPENDIX B-II  
DATA RETRIEVAL PROGRAMS



1. The following program is to retrieve the geotechnical information in the area of Township 37 North (T37N) and Range 9 West (R9W) then to put it onto the file in terms of PFILES VIKING 8

```

12345,ACC,CH150000,L10000,T400,TU30000,TC300,F20,TP1.
PASS=CDEF
REQUEST(TAPE,123)
REHIND(TAPE)
COPYDEF(TAPE,PROG)
RETURN(TAPE)
REHIND(PROG)
COMMON(SPSS)
SPSS(G=PROG,A=VIKING3)
REHIND(VIKING3)
PFILES(PUT,VIKING3)
7/8/9
GET FILE          PROG
SELECT IF         (TOWN EQ 37 AND TOWNDI EQ 1 AND RANGE EQ 9 AND RANGDI EQ 2)
WRITE CASES       (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
                   1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  A5,A3,F3.0,1X,A3,
                   F5.0, 1X,A2,F3.0,A1, 1X,A3, 1X,F3.0, 1X,F1.0/
                   11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A3,A2, 1X,F2.0, 1X,F2.0, 1X, A3,
                   1X,F5.1, 1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
                   11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
                   1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
                   1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 11X,
                   F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,
                   1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
                   11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
                   1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
                   1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
                   F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3-F4.2)
COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMD,DATEDAY,TOWN,TOWNDI,
RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTR,
CONTNO,ROADPR,ROADNO,ROADSU,CDRING,ASSOC,REFEAT,STATNO,OFFSET,
OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LAND,CRDSUR,DEPTHT,
DEPTH3,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,DEDRKS,
BEDRKS,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,FL,PI,SL,LOSSIG,
NATMC,NATMD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
MAXMD,OPTINC,CBRUN1,CBRUN2,CBRSD1,CBRSD2,CUSTR,AASHTO,UNIF,
CUSTA,TYPE,STRENGTH,STRAIN,CONFFRES,COHESION,ANGLE,POREPRES,
MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CU
FINISH

```





2. The following program is to present one-way frequency distribution tables for the variables as indicated in the program with the information out of PFILES VIKING 8

```

12345,ABC,MF150000,CM150000,L10000,P20,TU48000,TC480,T400.
PASS=CDEF
PFILES(GET,VIKING8)
COMMON(SPSS)
SPSS(D=VIKING8)
7/8/9
VARIABLE LIST COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,
RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
MAXWD,OPTMC,CBRUN1,CBRUN2,CBR01,CBR02,OUSTR,AASHTO,UNIF,
QUSTA,TYPE,STRENGTH,STRAIN,CONFRES,COHESION,ANGLE,POREPRES,
MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CU
N OF CASES 138
INPUT FORMAT (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0,1X,F2.0,1X,F2.0,
1X,F1.0,1X,F2.0,1X,F1.0,1X,F2.0,1X,A3,A5,A3,F3.0,1X,A3,
F5.0,1X,A2,F3.0,A1,1X,A8,1X,F3.0,1X,F1.0/
11X,F7.0,1X,F4.0,1X,F1.0,1X,A3,A2,1X,F2.0,1X,F2.0,1X,A8,
1X,FS.1,1X,F4.1,1X,F4.1,1X,F2.0,1X,F2.0,1X,F4.0/
11X,F2.0,1X,F1.0,1X,F1.0,1X,F1.0,1X,F3.1,1X,F4.1,1X,F2.1,
1X,F4.1,1X,F4.1,1X,F1.0,1X,F1.0,1X,F1.0,1X,F1.0,1X,F1.0,
1X,F2.0,1X,F4.1,1X,F4.1,1X,F4.1,1X,F4.1,1X,F4.1/ 11X,
F4.1,1X,F4.1,1X,F4.1,1X,F4.1,1X,F4.1,1X,F4.1,1X,F4.1,
1X,F4.1,1X,F4.1,1X,F4.1,1X,F4.1,1X,F3.1,1X,F3.1/
11X,F4.1,1X,F4.1,1X,F4.1,1X,F4.3,1X,F2.0,1X,F1.0,1X,F2.0,
1X,F2.0,1X,F4.1,1X,F4.1,1X,F3.1,1X,F3.1,1X,F3.1,1X,F3.1,
1X,F3.1,1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
FREQUENCIES GENERAL=SECTIO,ASSOC,GRDSUR,DEPTHT,DEPTHB,SPT,PHYSIO,SERIES,
PARENT,BEDRKS,BEDRKB,WATERF,PH,SAND,SILT,CLAY,COLL,LL,PL,PI,
SL,NATMC,NATDD,SPECGR,TEXTUR,ORGANI,MAXDD,OPTMC,CBR01,CBR02,
OUSTR,AASHTO,UNIF,COHESION,ANGLE,EO,PO,PC,CC,CR,CU
OPTIONS 5
FINISH

```



3. The following program is to present the distributions of textures, AASHTO classification units, Unified classification units, SPT values, and unconfined compressive strength values vs. depth with the information out of PFILES VIKING 8
- 

12345,ABC,MF150000,CM150000,L10000,P20,TU48000,TC480,T400.

PASS=CDEF

PFILES(GET,VIKING8)

COMMON(SPSS)

SPSS(D=VIKING8)

COPYCF(BCDOUT,,,RIB)

7/8/9

VARIABLE LIST COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,  
RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,  
CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,  
OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPHT,  
DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,  
BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,  
GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,  
NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,  
MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRSO1,CBRSO2,QUSTR,AASHTO,UNIF,  
QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,  
MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CU

N OF CASES 138

INPUT FORMAT (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,  
1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3, AS,A3,F3.0,1X,A3,  
F5.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/  
11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A8,A2, 1X,F2.0, 1X,F2.0, 1X, AS,  
1X,F5.1, 1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/  
11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,  
1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,  
1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/  
11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,  
1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/  
11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,  
1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,  
1X,F3.1, 1X,F4.2,F2.0,F2.0/  
11X F4.2,1X,F1.0,3F4.2,F3.2,  
F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.3,F4.2)

COMPUTE DEPTH=(DEPHT + DEPTHB)/2

SORT CASES DEPTH(A)

WRITE CASES (F6.2,SX,F2.0,SX,F2.0,SX,F2.0,SX,F2.0,SX,F5.2)DEPTH,TEXTUR,  
AASHTO,UNIF,SPT,QUSTR

FINISH



4. The following program is to present the CALCOMP plots of liquid limits, plastic limits, and natural moisture contents vs. depths and SPT values vs. depths with the information out of PFILES VIKING 8
- 

12345,ABC,MF150000,CM150000,L10000,P20,TU48000,TC480,T400.

PASS=CDEF

PFILES(GET,VIKING8)

COMMON(SPSS)

SPSS(D=VIKING8)

COPYPLT(NO PLOT)

7/8/9

RUN NAME WILD GEESSE

VARIABLE LIST COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,  
RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,  
CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,  
OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,  
DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,  
BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,  
GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,  
NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,  
MAXWD,OPTIMC,CBRUN1,CBRUN2,CBR501,CBR502,QUSTR,AASHTO,UNIF,  
QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,  
MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CU

N OF CASES 138

INPUT FORMAT (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,  
1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3, A5,A3,F3.0,1X,A3,  
F5.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/  
11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A8,A2, 1X,F2.0, 1X,F2.0, 1X, A8,  
1X,F5.1, 1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/  
11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,  
1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,  
1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/  
11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,  
1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/  
11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,  
1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,  
1X,F3.1, 1X,F4.2,F2.0,F2.0/  
11X F4.2,1X,F1.0,3F4.2,F3.2,  
F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)

COMPUTE

PLOT DEPTH=(DEPTHT + DEPTHB)/2  
PLOTS=LL,PL,NATMC(0.001,60) WITH DEPTH(0,140)/  
SIZE=-7.5,-4/  
XDIU=10/YDIU=5/  
SYMBOLS=1,2,0/  
PLOTS=SPT(0.001,80) WITH DEPTH(0,140)/  
SIZE=-7.5,-4/  
XDIU=10/YDIU=5/  
SYMBOLS=3/

FINISH



## APPENDIX C: MULTIPLE COMPARISON TABLES





Note:

<u>*VARIABLE NAME</u>	PHYSIO
<u>CODE</u>	<u>DESCRIPTION</u>
1.	Tipton Till Plain
2.	Dearborn Upland
3.	Muscatatuck Regional Slope
4.	Scottsburg Lowland
5.	Norman Upland
6.	Mitchell Plain
7.	Crawford Upland
8.	Wabash Lowland
9.	Calumet Lacustrine Section
10.	Valparaiso Moraine
11.	Kankakee Lacustrine Section
12.	Maumee Lacustrine Section
13.	Steuben Morainal Section

<u>**VARIABLE NAME</u>	AASHTO
<u>CODE</u>	<u>DESCRIPTION</u>
1.	A-1-A
2.	A-1-B
3.	A-2-4
4.	A-2-5
5.	A-2-6
6.	A-2-7
7.	A-3
8.	A-4
9.	A-5
10.	A-6
11.	A-7-5
12.	A-7-6
13.	A-8



\*\*\*VARIABLE NAME

ASSOC

CODEDESCRIPTION

1.	Eel-Martinsville-Genesee
2.	Genesee-Ross-Shoals
3.	Wakeland-Stendal-Haymond-Bartle
4.	Genesee-Shoals-Eel
5.	Haymond-Nolin-Petrolia
6.	Genesee-Eel-Stendal-Pope
7.	Huntington-Wheeling-Markland
8.	Huntington-Lindside
9.	Haymond-Wakeland
10.	Alida-Del Rey-Whitaker
11.	Bono-Maumee-Warners
12.	Chelsea-Hillsdale-Oshtemo
13.	Conrad-Wooten-Weiss
14.	Door-Tracy-Quinn
15.	Door-Lydick
16.	Elston-Wea
17.	Dubois-Otwell-Bartle
18.	Fox-Martinsville-Aluvi
19.	Fox-Nineveh-Ockley
20.	Fox-Rodman
21.	Fulton-Rimer-Milford-Rensselaer
22.	Homer-Sebawa-Gilford
23.	Maumee-Gilford-Rensselaer
24.	Maumee-Newton
26.	Martinsville-Bellmore-Fox
27.	Martinsville-Whitaker
28.	Mahalasville-Whitaker
30.	Milford-Montgomery-Rensselaer
31.	McGary
32.	Negley-Parke
33.	Oshtemo-Bronson
34.	Oakville-Plainfield-Adrian
35.	Oshtemo-Fox



*** <u>VARIABLE NAME</u>	ASSOC (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
36.	Ockley-Westland
37.	Ockley-Wea
38.	Ockley-Fox
39.	Plainfield-Brems-Morrocco
40.	Plainfield-Tyner-Oshtemo
41.	Plainfield-Watseka
42.	Plainfield-Chelsea
43.	Patton-Henshaw
44.	Patton-Lyles-Henshaw
45.	Peoga-Bartle-Hosmer
46.	Parke-Miami-Negley
47.	Rensselaer-Montgomery
48.	Rensselaer-Darroch
49.	Rensselaer-Whitaker
50.	Vincennes-Zipp-Ross
51.	Volinia
53.	Wea-Crane
54.	Warsaw-Elston-Fox
55.	Westland-Sleeth
56.	Weinbach-Sciotoville
57.	Weinbach-Wheeling
58.	Crosier-Brookston
59.	Brookston-Odell-Corwin
61.	Blount-Morley-Pewamo
62.	Blount-Pewamo
63.	Riddles-Miami-Crosier
64.	Crosby-Brookston
65.	Elliott-Markham-Pewamo
66.	Fincastle-Ragsdale-Brookston
67.	Hoytville-Nappanee
69.	Parr-Miami
70.	Parr-Corwin



<u>***VARIABLE NAME</u>	ASSOC (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
71.	Randolph-Millsdale
72.	Reesville-Ragsdale
73.	Raub-Ragsdale
74.	Ragsdale-Sidell
76.	Russell-Hennepin
77.	Russell-Xenia
78.	Miami-Metea-Celina
79.	Miami-Owosso-Riddles
80.	Miami-Crosier-Metea
81.	Miami-Russell-Fincastle
82.	Miami-Fox-Milton
83.	Miami-Crosby
84.	Miami-Hennepin
85.	Miami-Fox-Martinsville
86.	Morley-Blount
87.	Muskingum-Shadeland-High Gap
88.	Odell-Chalmers
89.	Sidell-Parr
90.	Hennepin-Rodman
91.	Avonburg-Clermont
92.	Cincinnati-Hickory
93.	Cincinnati-Rossmoyne-Hickory
95.	Cincinnati-Ava
96.	Cincinnati-Ava-Alford
98.	Crider-Hagerstown-Frederick
99.	Crider-Frederick
100.	Corydon-Weikert-Berks
101.	Fairmount-Switzerland
102.	Grayford
103.	Lawrence-Bedford-Crider
104.	Tilsit-Johnsburg
105.	Wellston-Zanesville-Berks





<u>***VARIABLE NAME</u>	ASSOC (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
106.	Berks-Gilpin-Weikert
107.	Zanesville-Wellston
108.	Mucks-Peats
109.	Alford
110.	Bloomfield-Princeton-Ayrshire
111.	Hosmer
112.	Iva-Ava
113.	Hosmer-Cincinnati-Iva
114.	Lyles-Ayrshire-Princeton
115.	Princeton-Ayrshire-Bloomfield
116.	Princeton-Fox

<u>****VARIABLE NAME</u>	SERIES
<u>CODE</u>	<u>DESCRIPTION</u>
10.	Ade
20.	Adrian
30.	Alford
40.	Algiers
50.	Alida
60.	Allison
70.	Armiesburg
80.	Aubbeenaubbee
90.	Ava
100.	Avonburg
110.	Ayr
120.	Ayrshire
130.	Bartle
140.	Baxter
150.	Bedford
160.	Bellmore
170.	Berks



<u>****VARIABLE NAME</u>	SERIES (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
180.	Birds
190.	Bloomfield
200.	Blount
210.	Bonnie
220.	Bono
230.	Boonesboro
240.	Boyer
250.	Brady
260.	Brems
270.	Bronson
280.	Brookston
290.	Burgin
300.	Burnside
310.	Camden
320.	Carlisle
330.	Casco
340.	Catlin
350.	Celina
360.	Chalmers
370.	Chelsea
380.	Cincinnati
390.	Clarence
400.	Clermont
410.	Colyer
420.	Conover
430.	Conrad
440.	Corwin
450.	Cory
460.	Corydon
470.	Coupee
480.	Crane
490.	Crider



<u>****VARIABLE NAME</u>	SERIES (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
500.	Crosby
510.	Crosier
520.	Cuba
530.	Dana
540.	Darroch
550.	Del Rey
560.	Dickinson
570.	Door
580.	Dowagiac
590.	Dubois
600.	Dunning
610.	Eden
620.	Edenton
630.	Edwards
640.	Eel
650.	Elkinsonville
660.	Elliott
670.	Elston
680.	Evansville
690.	Fabius
700.	Fairmount
710.	Fincastle
720.	Flanagan
730.	Foresman
740.	Fox-Silt Loam
741.	Fox-Loam
742.	Fox-Urban Land
750.	Frederick
760.	Fulton
770.	Genesee-Silt Loam
771.	Genessee-Urban Land
780.	Gilford
790.	Gilpin



<u>***VARIABLE NAME</u>	SERIES (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
800.	Ginat
810.	Glenhall
820.	Granby
830.	Grayford
840.	Guthrie
850.	Hagerstown
860.	Hanna
870.	Haskins
880.	Haubstadt
890.	Haymond
900.	Hennepin
910.	Henshaw
920.	Hickory
930.	High Gap
940.	Hillsdale
950.	Homer
960.	Hoopeston
970.	Hosmer
980.	Houghton
990.	Hoytville
1000.	Huntington
1010.	Huntsville
1020.	Iona
1030.	Ipava
1040.	Iva
1050.	Jasper
1060.	Jennings
1070.	Johnsburg
1080.	Jules
1090.	Kalamazoo
1100.	Kerston
1110.	Kings





**** <u>VARIABLE NAME</u>	SERIES (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
1120.	Kokomo
1130.	Landes
1140.	Lawrence
1150.	Lenawee
1160.	Lindside
1170.	Linkville
1180.	Longlois
1190.	Lorenzo
1200.	Lowell
1210.	Lucas
1220.	Lydick
1230.	Lyles
1240.	Mahalasville
1250.	Markham
1260.	Markland
1270.	Martinsville
1280.	Martisco
1290.	Massie
1300.	Matherton
1310.	Maumee
1320.	McGary
1330.	Medway
1340.	Mellott
1350.	Mermill
1360.	Metamora
1370.	Metea
1380.	Miami-Silt Loam
1381.	Miami-Urban Land
1390.	Milford
1400.	Millsdale
1410.	Milton
1420.	Monitor



<u>****VARIABLE NAME</u>	SERIES (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
1430.	Montgomery
1440.	Montmorenci
1450.	Morley
1460.	Morocco
1470.	Muren
1480.	Muskingum
1490.	Mussey
1500.	Nappanee
1510.	Negley
1520.	Newark
1530.	Newton
1540.	Nicholson
1550.	Nineveh
1560.	Nolin
1570.	Oakville
1580.	Ockley
1590.	Octagon
1600.	Odell
1610.	Oshtemo
1620.	Otwell
1630.	Owosso
1640.	Palms
1650.	Parke
1660.	Parr
1670.	Patton
1680.	Pekin
1690.	Peoga
1700.	Petrolia
1710.	Pewamo
1720.	Pike
1730.	Pinhook
1740.	Plainfield



<u>****VARIABLE NAME</u>	<u>SERIES (continued)</u>
<u>CODE</u>	<u>DESCRIPTION</u>
1750.	Plano
1760.	Pope
1770.	Princeton
1780.	Proctor
1790.	Quinn
1800.	Ragsdale
1810.	Rahm
1820.	Randolph
1830.	Rarden
1840.	Raub
1850.	Rawson
1860.	Reesville
1870.	Rensselaer
1880.	Riddles
1890.	Rimer
1900.	Robinson
1910.	Rockcastle
1920.	Rodman
1930.	Ross
1940.	Rossmoyne
1950.	Runnymede
1960.	Rush
1970.	Russell
1980.	St. Clair
1990.	Saranac
2000.	Saugatuck
2010.	Sciotoville
2020.	Sebewa
2030.	Seward
2040.	Shadeland
2050.	Shipshe
2060.	Shoals



<u>****VARIABLE NAME</u>	<u>SERIES (continued)</u>
<u>CODE</u>	<u>DESCRIPTION</u>
2070.	Sidell
2080.	Sleeth
2090.	Sloan
2100.	Sparta
2120.	Starks
2120.	Steff
2130.	Stendal
2140.	Stonelick
2150.	Stoy
2160.	Strole
2170.	Sunbury
2180.	Switzerland
2190.	Swygert
2200.	Sylvan
2210.	Taggart
2220.	Tama
2230.	Tedrow
2240.	Tilsit
2250.	Tippecanoe
2260.	Toledo
2270.	Toronto
2280.	Tracy
2290.	Trappist
2300.	Troxel
2310.	Tyner
2320.	Uniontown
2330.	Vigo
2340.	Vincennes
2350.	Volinia
2360.	Wakeland
2370.	Wallkill
2380.	Warners
2390.	Warsaw





**** <u>VARIABLE NAME</u>	SERIES (continued)
<u>CODE</u>	<u>DESCRIPTION</u>
2400.	Wasepi
2410.	Washtenaw
2420.	Watseka
2430.	Wauseon
2440.	Wea
2450.	Weikert
2460.	Weinbach
2470.	Wellston
2480.	Westland
2490.	Wheeling
2500.	Whitaker
2510.	Whitson
2520.	Wilbur
2530.	Willette
2540.	Wingate
2550.	Woodmere
2560.	Woolper
2570.	Wynn
2580.	Xenia
2590.	Zanesville
2600.	Zipp
2700.	Borrow Pit
2800.	Urban Land
2900.	Alluvial Land
3000.	Gravel Pit
3100.	Gullied Land
3200.	Strip Mine
3300.	Cut and Fill
3400.	Made Land



JHRP 80/7

CONTRACT  
PROJECT  
DISTRICT

CAL

No.	WORK ITEM
1	Move In
2	Remove Existing Struc
3	Order and Deliver Piling
4	Construct Fill
5	Bent 1 Cofferdam
6	Bent 1 Piling
7	Bent 1 Form & Pour Footi
	Bent 1

JHRP 80/7 3 Sheets

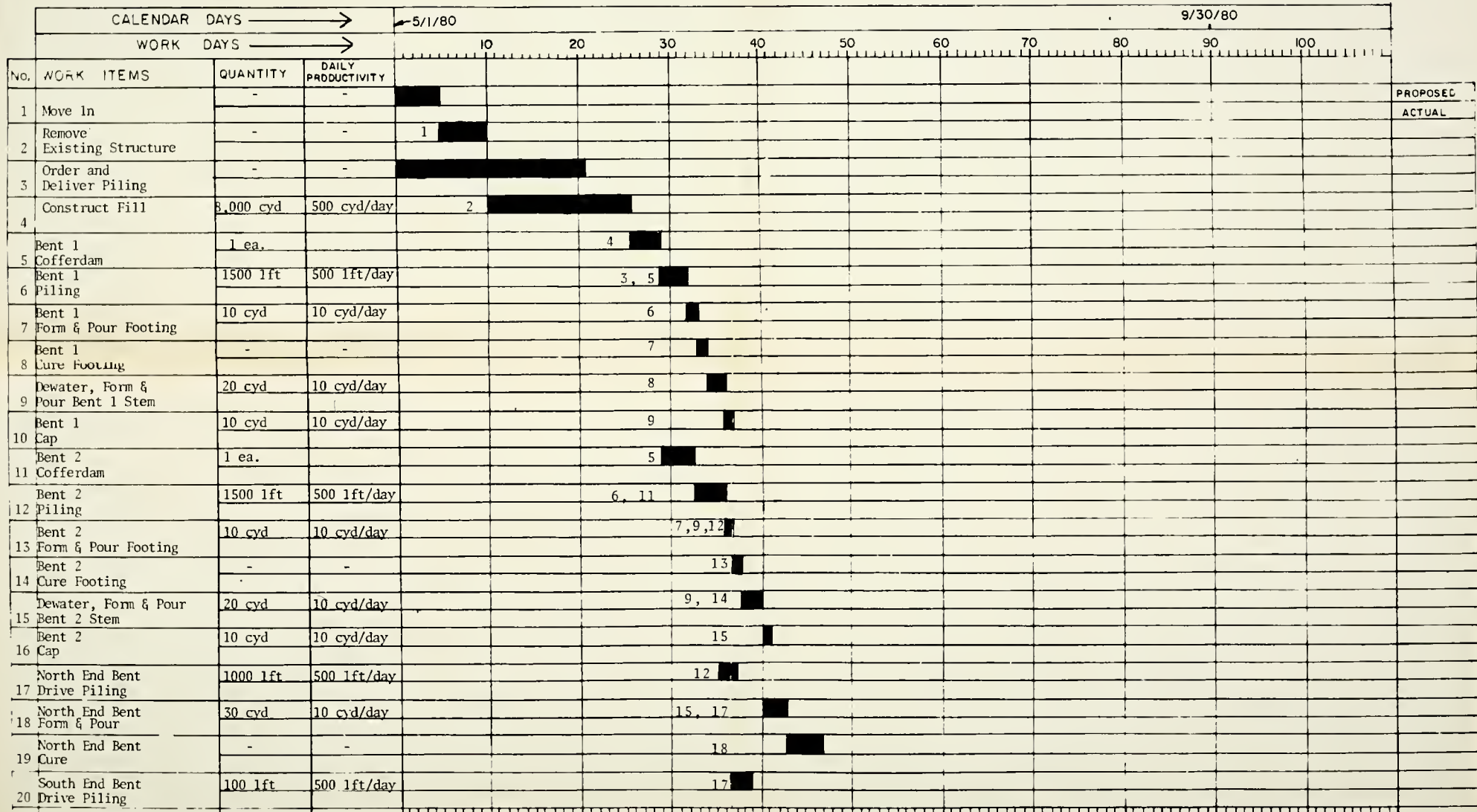


## INDIANA STATE HIGHWAY COMMISSION

CONTRACT Bridge Project - Sheet 1  
 PROJECT                       
 DISTRICT                     

LETTING DATE                       
 START DATE May 1, 1980  
 COMPLETION DATE                     

MADE BY                       
 CHECKED BY                       
 APPROVAL BY                     

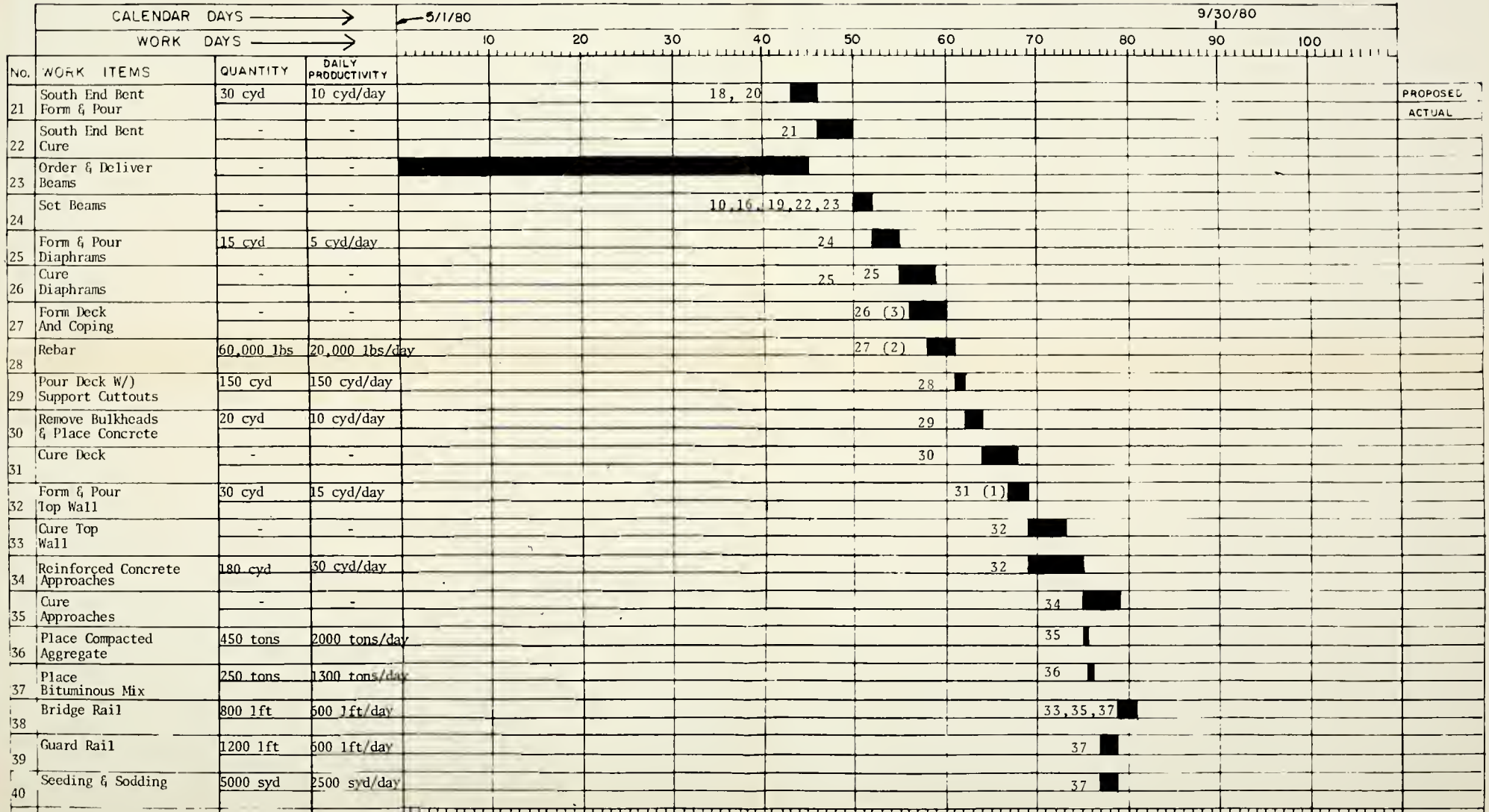


## INDIANA STATE HIGHWAY COMMISSION

CONTRACT Bridge Project - Sheet 2  
PROJECT \_\_\_\_\_  
DISTRICT \_\_\_\_\_

LETTING DATE \_\_\_\_\_  
START DATE \_\_\_\_\_  
COMPLETION DATE \_\_\_\_\_

MADE BY \_\_\_\_\_  
CHECKED BY \_\_\_\_\_  
APPROVAL BY \_\_\_\_\_



LETTING DATE \_\_\_\_\_  
START DATE \_\_\_\_\_  
COMPLETION DATE \_\_\_\_\_

MADE BY \_\_\_\_\_  
CHECKED BY \_\_\_\_\_  
APPROVAL BY \_\_\_\_\_

[illegible]



COVER DESIGN BY ALDO GIORGINI